REPORT TO

AVEO GROUP

ON

GEOTECHNICAL INVESTIGATION AND STABILITY ASSESSMENT
(In Accordance with Pittwater Council Risk Management Policy)

FOR

PROPOSED BUSH LIFT

AT

BAYVIEW GARDENS RETIREMENT LIVING, 36-42 CABBAGE TREE ROAD, BAYVIEW, NSW

Date: 16 October 2024

Ref: 37080Yrpt2

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1 INTRODUCTION

This report presents the results of a geotechnical investigation and assessment for the proposed bush lift at Bayview Gardens Retirement Village, 36-42 Cabbage Tree Road, Bayview, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Jessica Gleeson by signed 'Acceptance of Proposal' form dated 26 September 2024 and was carried out on the basis of our fee proposal, (Ref: P70180YF, dated 13 September 2024).

Based on the provided architectural drawings prepared by Bokor Architecture (Project No. 22043, Relevant Drawing Nos. DA-000, DA-011 and DA-402, all Revision 2, dated 24 July 2024), we understand the proposed bush lift will be constructed adjacent to the existing storeroom and will be excavated into the hillside such that it will be used to connect the pedestrian pathway to the road to the south-west and north-east, respectively. As such, excavation to depths ranging from about 1.4m to 2.6m will be required at the south-western and north-eastern ends of the lift, respectively. A suspended walkway will extend from the lift to the north-east and will connect to the existing pathway junction.

Based on Council's Geotechnical Hazard Mapping the site is located within the Council Geotechnical Hazard Zone H1, and therefore, a geotechnical slope stability assessment is required.

This report has been prepared in accordance with the requirements of the Geotechnical Risk Management Policy for Pittwater (2009) as discussed in Section 5 below. We understand that the development already has DA approval from council, but for completeness, we have completed Council Forms 1 and 1a.

2 ASSESSMENT & INVESTIGATION METHODOLOGY

2.1 Walkover Survey

This stability assessment is based upon a detailed inspection of the topographic, surface drainage and geological conditions of the site and its immediate environs. The attached Appendix A defines the terminology adopted for the risk assessment together with a flowchart illustrating the Risk Management Process based on the guidelines given in AGS 2007c (Reference 1).

A summary of our observations is presented in Section 3 below. Our specific recommendations regarding the proposed development are discussed in Section 6 following our geotechnical assessment.

The attached Figure 3 presents a geotechnical sketch plan showing the principal geotechnical features identified on site, as well as the investigation locations. Figure 3 is based on the survey plan prepared by CMS Surveyors (Ref: 15880Edetail, Sheet 3, Issue 7, dated 7 June 2024). Additional features on Figure 3 have been measured by hand held clinometer and tape measure techniques and hence are only approximate. Should any of the features be critical to the proposed development, we recommend they be located more accurately using instrument survey techniques. Figure 4 presents the geotechnical sketch plan with the assessed geotechnical hazards. Figure 5 presents a cross-section through the site based on the survey data augmented by our mapping observations, and also show the geotechnical hazards. Figure 6 defines the mapping symbols used.

2.2 Subsurface Investigation

Due to the limited site access, the investigation was carried out using hand operated equipment. The fieldwork was carried out on 2 October 2024 and comprised the following:

- Two hand auger boreholes, BH1 and BH4, were drilled to 0.85m and 0.35m below existing surface levels, respectively, at which depth refusal occurred. The boreholes were used to identify the materials present.
- Five Dynamic Cone Penetration (DCP) tests, DCP1 to DCP5, were completed adjacent to the borehole locations (DCP1 and DCP4) and at three additional locations (DCP2, DCP3 and DCP5). The DCP tests extended to refusal depths ranging from 0.65m to 1.35m. The purpose of the DCP tests was to provide an assessment of the apparent compaction of the fill and the relative density of the natural soils and attempt to probe to the surface of the underlying bedrock. The refusal depth of the DCP tests may provide an indicative depth to rock, although we note that premature refusal can also occur on obstructions in fill, 'floaters' and other hard layers.
- One test pit, TP1, was excavated on the south-eastern side of the existing storeroom to a depth of about 0.3m below existing ground levels. The test pit was excavated in an attempt to assess the dimensions of the footing and the materials on which it is founded.

The test locations, as shown on the attached Figure 2, were set out by tape measurements from existing surface features. The approximate surface levels at the test locations have been interpolated from spot heights shown on the survey plan prepared by CMS Surveyors (Ref: 15880Edetail, Sheet 3, Issue 7, dated 7 June 2024). The height datum is Australian Height Datum.

Groundwater observations were made in the boreholes during and on completion of drilling. No longer term groundwater monitoring was completed.

The fieldwork was completed in the full-time presence of our senior geotechnical engineer, Mr Ben Sheppard, who set out the investigation locations, nominated the sampling and testing, prepared the borehole logs and recorded the DCP test results. The borehole logs and DCP test results are attached with this report, together with a set of explanatory notes which define the logging terms and symbols used and provide further details of the investigation techniques adopted and their limitations.

Selected soil samples were returned to NATA accredited laboratory (Envirolab Services Pty Ltd) for soil pH, chloride content, sulphate content and resistivity testing. These results are summarised in the attached Envirolab Services Certificate of Analysis 363219.

3 SUMMARY OF OBSERVATIONS

We recommend that the summary of observations which follows be read in conjunction with the attached Figures 1 to 5.

The site is located about mid-slope on a south and south-west sloping hillside, which slopes down from an east-west orientated spur to a low-lying alluvial gully associated with Pittwater Bay. The site is located within Bayview Gardens Retirement Village, which generally comprises several one to three storey structures, internal Asphaltic Concrete (AC) paved roads, pathways, gardens and lawns.

For the purpose of this report, the 'site' is limited to the area to the south-east of the existing storeroom, as shown in the below Plate 1. The site comprises several elevation changes, with an overall slope to the south-west at about 30°.

Plate 1 – Existing Site Conditions, looking north-east

The site generally comprises a sandstone boulder wall, which has a total height of about 3.8m and an average slope of about 30° down to the south-west. The sandstone boulders were generally ‘lenticular’ in shape and have an average size of about 1m x 0.8m x 0.5m, although some larger and smaller boulders were also evident. Along the crest of the boulder wall is an AC surfaced road, which generally appears to be in good condition, with the exception of some longitudinal cracking (parallel to the embankment), as shown in the below Plate 2. A sewer manhole and subsequent spray paint marking on the road indicates the presence of a sewer which runs near the edge of the boulder wall at depth of about 2.9m below existing road levels

Plate 2 – Above Boulder wall, Looking north-west

A single storey brick storeroom is located along the north-western side of the boulder wall, and appears to have been built into the hillside along its eastern edge. Above the storeroom and extending to the north-west is the ‘bridgehouse’, which forms the landing structure for the timber bridge which traverses the creek located at the toe of the slope. Along the toe of the boulder wall is a concrete footpath, which is generally level and appears to be in fair condition. The south-western edge of the footpath is supported by a sandstone boulder wall, which is about 1.2m high and slopes to the south-west at between about 40° to near vertical, as shown in the below Plate 3. The pathway was undermined by up to about 0.2m in some sections. Some erosion/washing of the soils between the boulders was evident. To the north of the lower boulder wall was a sandstone block retaining wall, which varies in height between about 0.8m and 2m, and generally appeared to be in good external condition.

Plate 3 – Lower boulder wall, looking north-east

At the toe of the lower boulder wall was a vegetated hillside which slopes at about 20° to 30° towards a creek. Towards the lower reaches of the slope, adjacent to the creek, medium to large trees appeared to exhibit signs basal curvature, with one large tree curving at up to about 40° from the horizontal near its base, as shown in the below Plate 4.

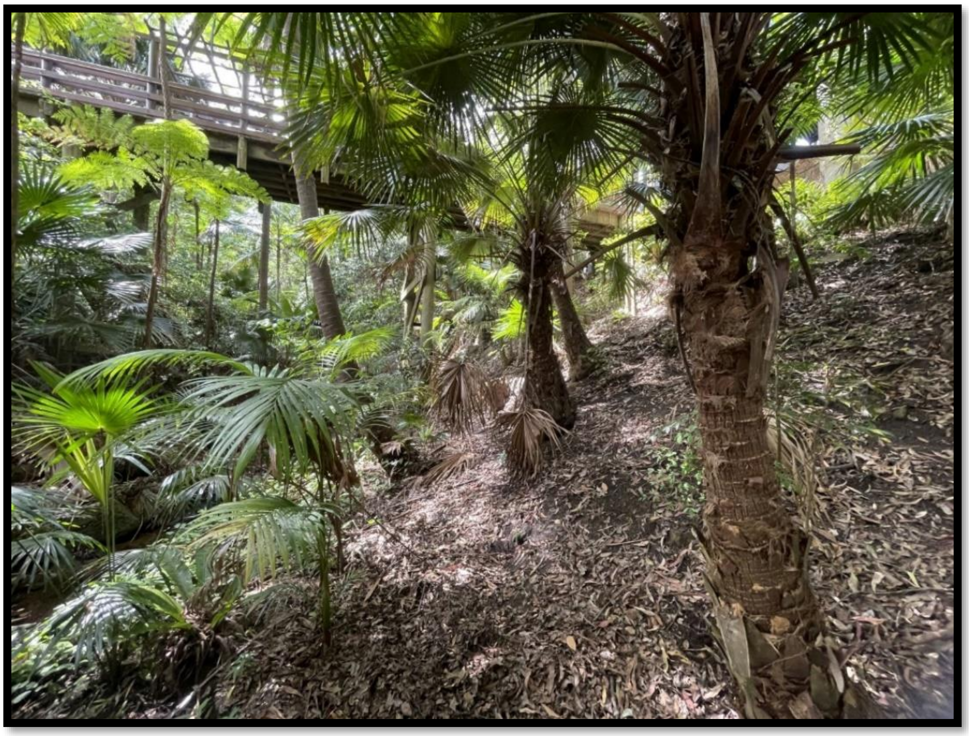

Plate 4 – Hillside adjacent to creek, looking north-west

4 SUBSURFACE CONDITIONS

The 1:100,000 geological map of Sydney indicates that the site is underlain by the Newport Formation of the 'Narrabeen Group', which comprises interbedded laminite, shale and quartz to lithic quartz sandstone. The upper portions of the ridgeline to the north of the site are mapped to comprise Hawksbury Sandstone.

The boreholes and test pit generally encountered fill overlying inferred weathered bedrock. The fill was variable and comprised silty sand, silty clay and sandy silty clay and based on the DCP test results, was assessed to be poorly compacted. Inclusions within the fill comprised sandstone and ironstone gravel.

The DCP tests refused at depths ranging from 0.35m (DCP3) and 3.5m (DCP4A). With the exception of DCP3, the refusal depths of the DCP tests have been inferred to have occurred on the surface of the underlying weathered bedrock. DCP3 appears to have refusal on the back of one of the boulders from the boulder wall. However, since these tests do not provide sample recovery this could not be confirmed and it is possible that refusal may have occurred on obstructions within the fill or other hard layers.

No groundwater was encountered within the boreholes during or on completion of drilling.

Reference should be made to the boreholes logs, DCP test results and test pit sketch for detailed descriptions of the subsurface conditions encountered.

Test Pit

The test pit excavated adjacent to the storeroom indicated that the 0.23m brick wall appears to be founded directly on the what appears to be a sandstone cobble at a depth of about 0.1m. This cobble appears to be located within a sandy silty clay fill matrix, which appears to be poorly compacted.

Laboratory Test Results

The results of the soil aggression testing are tabulated below:

Borehole	Depth (m)	Sample Type	pH	Sulphates SO_4 (ppm)	Chlorides CL (ppm)	Resistivity (ohm.cm)
BH1	0.3 – 0.5	Sandy Silty CLAY	7.2	30	55	7,700

The above results indicate that for concrete and steel structures in contact with the ground, the fill has an exposure classification of 'Non-Aggressive' when assessed in accordance with Tables 6.4.2 (C) and 6.5.2 (C) of AS2159-2009 "Piling Design and Installation".

5 GEOTECHNICAL ASSESSMENT

We consider that the potential landslide hazards associated with the site to be the following:

A. Instability of 3.6m high sandstone boulder wall.
B. Instability of 1.2m high sandstone boulder wall.
C. Shallow failure of slope.
D. Deep seated failure of slope.
E. Instability of sandstone block retaining wall.
G. Instability of proposed retaining walls to support soils and bedrock for the proposed excavation.

These potential hazards are indicated in schematic form on the attached Figure 4 and 5.

5.3 Risk Analysis

The attached Table A summarises our qualitative assessment of each potential landslide hazard and of the consequences to property should the landslide hazard occur. Use has been made of data in MacGregor *et al* (2007) to assist with our assessment of the likelihood of a potential hazard occurring. Based on the above, the qualitative risks to property have been determined. The terminology adopted for this qualitative assessment is in accordance with Table A1 given in Appendix A. Table A indicates that the assessed risk to property varies between Low or Very Low, which would be considered 'acceptable' in accordance with the criteria given in Reference 1 and the Pittwater Council Risk Management Policy.

We have also used the indicative probabilities associated with the assessed likelihood of instability to calculate the risk to life. The temporal and vulnerability factors that have been adopted are given in the attached Table B together with the resulting risk calculation. Our assessed risk to life for the person most at risk is about 5×10^{-7}, which would be considered to be 'acceptable' in relation to the criteria given in Reference 1 and the Pittwater Council Risk Management Policy.

5.4 Risk Assessment

The Pittwater Risk Management Policy requires suitable measures 'to remove risk'. It is recognised that, due to the many complex factors that can affect a site, the subjective nature of a risk analysis and the imprecise nature of the science of geotechnical engineering, the risk of instability for a site and/or development cannot be completely removed. It is, however, essential that risk be reduced to at least that which could be reasonably anticipated by the community in everyday life and that landowners are made aware of reasonable and practical measures available to reduce risk as far as possible. Hence, where the policy requires that 'reasonable and practical measures have been identified to remove risk', it means that there has been an active process of reducing risk, but it does not require the geotechnical engineer to warrant that risk has been completely removed, only reduced, as removing risk is not currently scientifically achievable.

Similarly, the Pittwater Risk Management Policy requires that the design project life be taken as 100 years unless otherwise justified by the applicant. This requirement provides the context within which the geotechnical risk assessment should be made. The required 100 years baseline broadly reflects the expectations of the community for the anticipated life of a residential structure and hence the timeframe to be considered when undertaking the geotechnical risk assessment and making recommendations as to the appropriateness of a development, and its design and remedial measures that should be taken to control risk. It is recognised that in a 100 year period external factors that cannot reasonably be foreseen may affect the geotechnical risks associated with a site. Hence, the Policy does not seek the geotechnical engineer to warrant the development for a 100 year period, rather to provide a professional opinion that foreseeable geotechnical risks to which the development may be subjected in that timeframe have been reasonably considered.

Our assessment of the probability of failure of existing structural elements such as retaining walls (where applicable) is based upon a visual appraisal of their type and condition at the time of our inspection. Where existing structural elements such as retaining walls will not be replaced as part of the proposed development, where appropriate we identify the time period at which reassessment of their longevity seems warranted.

In preparing our recommendations given below we have adopted the above interpretations of the Risk Management Policy requirements. We have also assumed that no activities on surrounding land which may affect the risk on the subject site would be carried out. We have further assumed that all Council's buried services are, and will be regularly maintained to remain, in good condition.

We consider that our risk analysis has shown that the site and existing and proposed development can achieve the 'Acceptable Risk Management' criteria in the Pittwater Risk Management Policy provided that the recommendations given in Section 6 below are adopted. These recommendations form an integral part of the Landslide Risk Management Process.

6 COMMENTS AND RECOMMENDATIONS

We consider that the primary geotechnical issues for the proposed development to be:

- Maintaining support to the existing storeroom structure during excavation of the lift.
- Maintaining temporary stability of the excavation during construction.
- Ensuring sufficient bearing stratum is reached for the new footings so that they are uniformly founded within the weathered rock to reduce the risk posed by potential landslip instability.

6.1 Conditions Recommended to Establish the Design Parameters

6.1.1 All proposed footings must be founded in bedrock. The footings should be designed for an allowable bearing pressure of 600kPa, subject to inspection by a geotechnical engineer prior to pouring. Should footings be founded at the crest of excavations (i.e. where the storeroom is underpinned next to the lift pit excavation), the bedrock must be of greater than low strength and free from adverse defects.

6.1.2 The proposed lift pit excavation will extend adjacent to and below the existing storeroom structure. Therefore, all excavation works must be carried out with care to ensure that the existing footings are not undermined or rendered unstable. The excavated test pit revealed the structure to be supported possibly on a sandstone cobble founded within poorly compacted fill. These details must be confirmed in the initial stages of construction by the excavation of test pits to expose the existing footing so that an assessment of the most appropriate means of support to the storeroom may be confirmed. Such support may comprise underpinning to below the base of the proposed lift pit excavation or construction of a retention system prior to the commencement of excavation. The test pits will need to be inspected by the geotechnical engineer. As excavation is required to a maximum depth of ABOUT 2.6m, should good quality bedrock not be encountered within about 1m, underpinning will be more difficult and the installation of a retention system may be the better option.

6.1.3 Where space exists (i.e. to the to the south-east) temporary batters may be adopted. Subject to inspection by a geotechnical engineer temporary batters for the proposed excavation should be no steeper than 1 Vertical (V) in 1.5 Horizontal (H) within the soil profile and extremely weathered rock and vertical in competent rock. Surcharge loads, including construction loads, vehicles, stockpiles, etc. must be kept well clear of the crest of temporary batters (at least 2H from the crest, where H is the vertical height of the batter slope in metres). All loose sandstone boulders should be removed from the crest of the batters.

6.1.4 Where the required batters cannot be accommodated within the site geometry, or where not preferred, a retention system would be required and should be installed prior to excavation commencing. We recommend the retention system comprise a cantilevered contiguous pile wall. Design parameters are provided in Section 6.1.6 below.

6.1.5 The surface water discharging from the new roof and paved areas must be diverted to outlets for controlled discharge to the existing stormwater system which appears to drain to creek.

6.1.6 The proposed new retaining walls should be designed using the following parameters:

- For cantilever walls, adopt a triangular lateral earth pressure distribution and where movement sensitive structures are not present within the zone of influence of the excavation (defined as a horizontal distance extending 2H from the crest of the wall, where H is the height of retained materials) and minor movements may be tolerated, an 'active' earth pressure coefficient, K_a, of 0.35, for the retained height, assuming a horizontal backfill surface.
- Where walls are temporarily propped, backfilled and permanently restrained by the structure, we recommend that they are designed using a triangular lateral earth pressure distribution using at an 'at rest' coefficient of lateral earth pressure, K_o, of 0.6, assuming a horizontal backfill surface.
- A bulk unit weight of 20kN/m^3 should be adopted for the soil profile.
- Any surcharge affecting the walls (eg. traffic loading, live loading, compaction stresses, etc) should be allowed in the design.
- The retaining walls for the lift pit must be designed as fully tanked.
- Toe resistance of the wall may be achieved by keying the footing into bedrock. An allowable lateral stress of 200kPa may be adopted for design for that portion of the socket founded within very low strength bedrock. The upper 0.5m of the socket formed below BEL should be ignored due to excavation disturbance and any localised deeper excavations. Higher lateral pressures may be adopted where higher strength rock is proven. Care is required not to over-excavate in front of the piles, and all excavations in front of the walls, such as for footings, buried services, etc. must be taken into account in the wall design.

6.1.7 The guidelines for Hillside Construction given in Appendix B should also be adopted.

6.2 Conditions Recommended to the Detailed Design to be Undertaken for the Construction Certificate

6.2.1 An excavation/retention methodology must be prepared prior to bulk excavation commencing. The methodology must include but not be limited to proposed excavation techniques, the proposed excavation equipment, excavation sequencing, geotechnical inspection intervals or hold points, vibration monitoring procedures, monitor locations, monitor types, contingency plans in case of exceedances.

6.2.2 If underpinning is required, a detailed underpinning methodology must be developed by the structural engineer and reviewed by the geotechnical engineer prior to commencing such works. Any underpinning should be carried out by the excavation of discrete sections with each section fully underpinned prior to excavation of the adjacent sections. Regular geotechnical and structural inspections would be required during the underpinning works. Where bedrock of better than low strength that is free from adverse defects is not encountered at relatively shallow depth, underpinning is likely to be difficult and many need to be completed in stages, with underpins initially extended to a depth of about 1.3m, excavation then extended to a depth of about 1m before these underpins in turn are underpinned to below bulk excavation level. Careful consideration must be given to these issues when developing the design.

6.2.3 All structural design drawings must be reviewed by the geotechnical engineer who should endorse that the recommendations contained in this report have been adopted in principle.

6.2.4 All hydraulic design drawings must be reviewed by the geotechnical engineer who should endorse that the recommendations contained in this report have been adopted in principle.

6.2.5 All landscape design drawings must be reviewed by the geotechnical engineer who should endorse that the recommendations contained in this report have been adopted in principle.

6.2.6 The excavation/retention methodology must be reviewed and approved by the geotechnical engineer.

6.3 Conditions Recommended During the Construction Period

6.3.1 The approved excavation/retention methodology must be followed.

6.3.2 The geotechnical and structural engineers should inspect test pits excavated adjacent to the storeroom and confirm the founding conditions and advise on the underpinning methodology.

6.3.3 While site access is restricted, we anticipate that access for a small tracked excavator may be feasible. As such, excavation of the soils and weathered bedrock of up to very low strength could be carried out using a small excavator equipped with buckets. The large boulders will require removal and may require the use of hydraulic hammers to break down large pieces into smaller manageable pieces. Where bedrock of low strength or stronger is encountered hydraulic hammers are likely to be required in its removal. Consequently, where hydraulic hammers are used, continuous vibration monitoring must be carried out to assess whether transmitted vibrations fall within acceptable limits, which is considered to be a peak particle velocity (PPV) of $5mms^{-1}$. Where transmitted vibrations exceed these limits, smaller hammers or non-vibratory excavation techniques must be adopted.

6.3.4 The geotechnical engineer should witness the drilling of representative retention piles to confirm that the design criteria are satisfied.

6.3.5 The geotechnical engineer should inspect all temporary batters.

6.3.6 The geotechnical engineer must inspect all footing excavations and pile hole drilling prior to placing reinforcement or pouring the concrete to confirm that the design ABP's have been reached.

6.3.7 Proposed material to be used for backfilling behind retaining walls must be approved by the geotechnical engineer prior to placement.

6.3.8 Compaction density of the backfill material must be checked by a NATA registered laboratory to at least Level 2 in accordance with, and to the frequency outlined in, AS3798, and the results submitted to the geotechnical engineer.

6.3.9 The existing stormwater system, sewer and water mains must be checked for leaks by using static head and pressure tests under the direction of the hydraulic engineer or architect, and repaired if found to be leaking.

6.3.10 The geotechnical engineer must inspect all subsurface drains prior to backfilling.

6.3.11 An 'as-built' drawing of all buried services at the site must be prepared (including all pipe diameters, pipe depths, pipe types, inlet pits, inspection pits, etc).

6.3.12 The geotechnical engineer must confirm that the proposed works have been completed in accordance with the geotechnical reports.

We note that all above Conditions must be complied with. Where this has not been done, it may not be possible for Form 3, which is required for the Occupation Certificate to be signed.

6.4 Conditions Recommended for Ongoing Management of the Site/Structure(s)

The following recommendations have been included so that the current and future owners of the subject property are aware of their responsibilities:

6.4.1 All existing and proposed surface (including roof) and subsurface drains must be subject to ongoing and regular maintenance by the property owners. In addition, such maintenance must also be carried out by a plumber at no more than five yearly intervals; including provision of a written report confirming scope of work completed (with reference to the 'as-built' drawing) and identifying any required remedial measures.

6.4.2 No cut or fill in excess of 0.5m (eg. for landscaping, buried pipes, retaining walls, etc), is to be carried out on site without prior consent from Pittwater Council.

6.4.3 Where the structural engineer has indicated a design life of less than 100 years then the structure and/or structural elements must be inspected by a structural engineer at the end of their design life; including a written report confirming scope of work completed and identifying the required remedial measures to extend the design life over the remaining 100 year period.

7 OVERVIEW

The recommendations presented in this report include specific issues to be addressed during the design and construction phase of the project. In the event that any of the advice presented in this report is not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

The subsurface conditions between the completed boreholes may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

This report provides advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

A waste classification is required for any soil and/or bedrock excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), General Solid, Restricted Solid or Hazardous Waste. Analysis can take up to seven to ten working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) could be expected. We strongly recommend that this requirement is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

Reference 1: Australian Geomechanics Society (2007c) *'Practice Note Guidelines for Landslide Risk Management'*, Australian Geomechanics, Vol 42, No 1, March 2007, pp63-114.

Reference 2: MacGregor, P, Walker, B, Fell, R, and Leventhal, A (2007) *'Assessment of Landslide Likelihood in the Pittwater Local Government Area'*, Australian Geomechanics, Vol 42, No 1, March 2007, pp183-196.

TABLE A

SUMMARY OF RISK ASSESSMENT TO PROPERTY

POTENTIAL LANDSLIDE HAZARD	A	B	C	D	E	F
	Instability of 3.6m high sandstone boulder wall	Instability of 1.2m high sandstone boulder wall	Shallow Failure of slope	Deep Seated Failure of Slope	Instability of sandstone block retaining wall	Instability of proposed retaining walls to support soils and bedrock for the proposed excavations.
Assessed Likelihood	Unlikely	Possible	Unlikely	Rare	Unlikely	Barely Credible
Assessed Consequence	Minor	Insignificant	Medium	Major	Insignificant	Major
Risk	Low	Very Low	Low	Low	Very Low	Very Low
Comments	Boulder wall generally appears to be in good condition, comprises suitably sized blocks and is raked back at about 30°. Failure of the boulder wall would result in repairs to the roadway above, pathway below and reinstatement of boulder wall.	Possible signs of previous instability of the wall observed, likely due to erosion of soils between boulders. Failure would result in repair to concrete pathway above and re- instatement of boulder wall.	Some basal curvature of one medium sized tree within the boulder wall observed. More obvious signs of basal curvature towards lower reaches of the hillside, far away from the site. Considering the depth to rock, a shallow slide within the soils below boulder wall appears possible, although considering overall slopes of about 30° is considered unlikely. Failure would result in repairs to the roadway above, pathway below, reinstatement of boulder wall and possible damage to storeroom, if not founded on bedrock.	Depth to rock suggests that the site is not likely to be affected by deep seated slope failure. Failure could either be deep seated failure within soils only above the bedrock, or complex failure which would comprise failure through the soils and bedrock. Failure could result in significant damage to the road, sewer, boulder wall, storeroom and concrete pathway.	Wall generally in good condition and retains a maximum height of about 2m. Wall supports paved areas and is not supporting structures. Failure would result in repairs required to wall and reinstatement of paved area.	Assumes all retaining walls are engineer designed and properly constructed. Failure of walls would impact road and surrounding areas.

TABLE B

SUMMARY OF RISK ASSESSMENT TO LIFE

POTENTIAL LANDSLIDE HAZARD	A	B	C	D	E	F
	Instability of 3.6m high sandstone boulder wall	Instability of 1.2m high sandstone boulder wall	Shallow Failure of Slope	Deep Seated Failure of Slope	Instability of sandstone block retaining wall	Instability of proposed retaining walls to support soils and bedrock for the proposed excavations.
Assessed Likelihood	Unlikely	Possible	Unlikely	Rare	Unlikely	Barely Credible
Indicative Annual Probability	10^{-4}	10^{-3}	10^{-4}	10^{-5}	10^{-4}	10^{-6}
Persons at risk	Person walking along footpath below	Person above the wall walking along footpath.	(i) Above – Persons within parked cars on road above boulder wall (ii) Below – Person walking along footpath	(i) Above – Persons within parked cars on road (ii) Below – Person walking along footpath	Person within paved areas above wall	Person within/accessing lift
Duration of Use of area Affected (Temporal Probability)	Say 1 minute/day 6.9×10^{-4}	Say 1 minute/day 6.9×10^{-4}	(i) Say 10 minutes/day 6.9×10^{-3} (ii) Say 1 minute/day 6.9×10^{-4}	(1) Say 10 minutes/day 6.9×10^{-3} (ii) Say 1 minute/day 6.9×10^{-4}	Say 1 minute/day 6.9×10^{-4}	Say 5 minutes/day 3.5×10^{-3}
Probability of not Evacuating Area Affected	1.0	0.5	(i) 1.0 (ii) 0.5	(i) 1.0 (ii) 0.8	0.1	1.0
Spatial Probability	1m length (~1x average boulder size) over total site length 3m, 1/3 = 0.33	1m length (~1x max height) over total site length 3m, 1/3 = 0.33	3m length (~1x height of soil above rock) over total site length 3m, 3/3 = 1	Deep seated landslip to impact >3m width which is approx. length of site = 1.0	2m length (~1x max height) over total site length 3m, 2/3 = 0.66	3m length (~1x total max retention height) over total width of 3m, 3/3 = 1.0
Vulnerability to Life if Failure Occurs Whilst Person Present	1.0	0.1	(i) 0.5 (ii) 1.0	(i) 0.8 (ii) 1.0	0.5	1.0
Risk for Person most at Risk	2.3×10^{-8}	1.1×10^{-8}	(i) 3.5×10^{-7} (ii) 3.5×10^{-8}	(i) 5.6×10^{-8} (ii) 5.6×10^{-9}	2.3×10^{-9}	3.5×10^{-9}
Combined total Risk	5×10^{-7}					

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BOREHOLE LOG

Borehole No.
1
1/1
BUSH LIFT

Client: AVEO GROUP
Project: PROPOSED BUSH LIFT
Location: 36-42 CABBAGE TREE ROAD, BAYVIEW, NSW

Job No.: 37080Y **Method:** HAND AUGER **R.L. Surface:** ≈ 19.2m
Date: 2/10/24 **Datum:** AHD
Plant Type: - **Logged/Checked by:** B.S./W.T.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLET-ION					REFER TO DCP TEST RESULTS SHEET	0			FILL: Silty sand, fine to medium grained, dark brown, trace of fine to medium grained sandstone gravel, clay nodules, and root fibres.	D			APPEARS POORLY COMPACTED
									FILL: Sandy silty clay, low to medium plasticity, orange brown and dark brown, fine grained sand, trace of fine to medium grained sandstone and ironstone gravel, and silty sand bands.	w<PL			
						0.5							
									FILL: Silty clay, medium plasticity, orange brown mottled brown, trace of fine grained ironstone gravel.	w≈PL			POSSIBLY RESIDUAL
									END OF BOREHOLE AT 0.85m				
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

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BOREHOLE LOG

Borehole No.
4
1/1
BUSH LIFT

Client: AVEO GROUP
Project: PROPOSED BUSH LIFT
Location: 36-42 CABBAGE TREE ROAD, BAYVIEW, NSW

Job No.: 37080Y **Method:** HAND AUGER **R.L. Surface:** ≈ 22.8m
Date: 2/10/24 **Datum:** AHD
Plant Type: - **Logged/Checked by:** B.S./W.T.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION					REFER TO DCP TEST RESULTS SHEET	0			FILL: Silty sand, fine grained, dark brown, trace of fine to coarse grained sandstone and ironstone gravel, clay nodules and root fibres.	M			APPEARS POORLY COMPACTED
						0.5			END OF BOREHOLE AT 0.35m				HAND AUGER REFUSAL 2 ATTEMPTS WERE MADE TO ADVANCE LAYER, HOWEVER REFUSAL OCCURED AT, OR PRIOR TO, 0.35m DEPTH
						1							
						1.5							
						2							
						2.5							
						3							
						3.5							

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DYNAMIC CONE PENETRATION TEST RESULTS

BUSH LIFT

Client:	AVEO GROUP
Project:	PROPOSED BUSH LIFT
Location:	36-42 CABBAGE TREE ROAD, BAYVIEW, NSW

Job No.	37080Y	Hammer Weight & Drop: 9kg/510mm
Date:	2-10-24	Rod Diameter: 16mm
Tested By:	B.S.	Point Diameter: 20mm

Test Location	1	2	3	Test Location			
Surface RL	≈19.2m	≈19.0m	≈21.5m	Surface RL			
Depth (mm)	Blows per 100mm Penetration			Depth (mm)	Blows per 100mm Penetration		
0 - 100	1	1	1	3000-3100			
100 - 200	2	2	2	3100-3200			
200 - 300	3	5	2	3200-3300			
300 - 400	6	5	8/50mm	3300-3400			
400 - 500	5	5	REFUSAL	3400-3500			
500 - 600	1	6		3500-3600			
600 - 700	3	4		3600-3700			
700 - 800	3	2		3700-3800			
800 - 900	7	2		3800-3900			
900 - 1000	18	4		3900-4000			
1000 - 1100	17/30mm	7		4000-4100			
1100 - 1200	REFUSAL	7		4100-4200			
1200 - 1300		13		4200-4300			
1300 - 1400		13		4300-4400			
1400 - 1500		14		4400-4500			
1500 - 1600		17		4500-4600			
1600 - 1700		24/90mm		4600-4700			
1700 - 1800		REFUSAL		4700-4800			
1800 - 1900				4800-4900			
1900 - 2000				4900-5000			
2000 - 2100				5000-5100			
2100 - 2200				5100-5200			
2200 - 2300				5200-5300			
2300 - 2400				5300-5400			
2400 - 2500				5400-5500			
2500 - 2600				5500-5600			
2600 - 2700				5600-5700			
2700 - 2800				5700-5800			
2800 - 2900				5800-5900			
2900 - 3000				5900-6000			

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

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DYNAMIC CONE PENETRATION TEST RESULTS

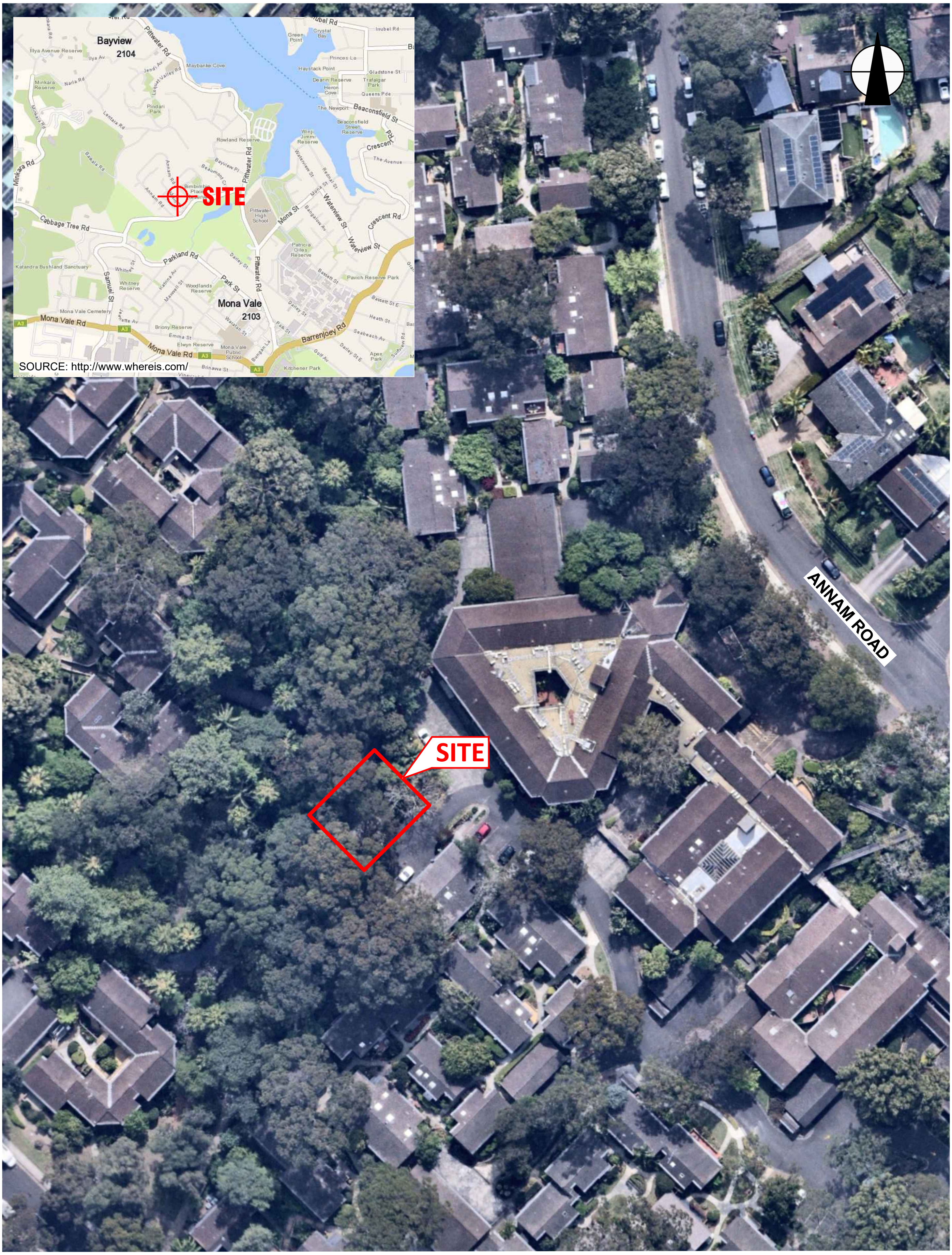
BUSH LIFT

Client:	AVEO GROUP
Project:	PROPOSED BUSH LIFT
Location:	36-42 CABBAGE TREE ROAD, BAYVIEW, NSW

Job No.	37080Y	Hammer Weight & Drop: 9kg/510mm
Date:	2-10-24	Rod Diameter: 16mm
Tested By:	B.S.	Point Diameter: 20mm

Test Location	**4**	**4A**	**5**	Test Location		**4A**	
Surface RL	≈22.8m	≈22.8m	≈17.0m	Surface RL		≈22.8m	
Depth (mm)	Blows per 100mm Penetration			Depth (mm)	Blows per 100mm Penetration		
0 - 100	1	1	1	3000-3100		4	
100 - 200	3	2	6	3100-3200		4	
200 - 300	2/50mm	3	3	3200-3300		8	
300 - 400	REFUSAL	2	6	3300-3400		8	
400 - 500		3	6	3400-3500		12	
500 - 600		4	5	3500-3600		8/0mm	
600 - 700		3	4	3600-3700		REFUSAL	
700 - 800		6	4	3700-3800			
800 - 900		7	4	3800-3900			
900 - 1000		6	3	3900-4000			
1000 - 1100		6	9	4000-4100			
1100 - 1200		5	10	4100-4200			
1200 - 1300		2	5	4200-4300			
1300 - 1400		2	4/40mm	4300-4400			
1400 - 1500		5	REFUSAL	4400-4500			
1500 - 1600		6		4500-4600			
1600 - 1700		6		4600-4700			
1700 - 1800		6		4700-4800			
1800 - 1900		6		4800-4900			
1900 - 2000		4		4900-5000			
2000 - 2100		5		5000-5100			
2100 - 2200		5		5100-5200			
2200 - 2300		3		5200-5300			
2300 - 2400		5		5300-5400			
2400 - 2500		3		5400-5500			
2500 - 2600		4		5500-5600			
2600 - 2700		6		5600-5700			
2700 - 2800		4		5700-5800			
2800 - 2900		5		5800-5900			
2900 - 3000		4		5900-6000			

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD



AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

This plan should be read in conjunction with the JK Geotechnics report.

Title:	**SITE LOCATION PLAN**
Location:	36-42 CABBAGE TREE ROAD, BAYVIEW, NSW
Report No: 37080YF	Figure No: 1

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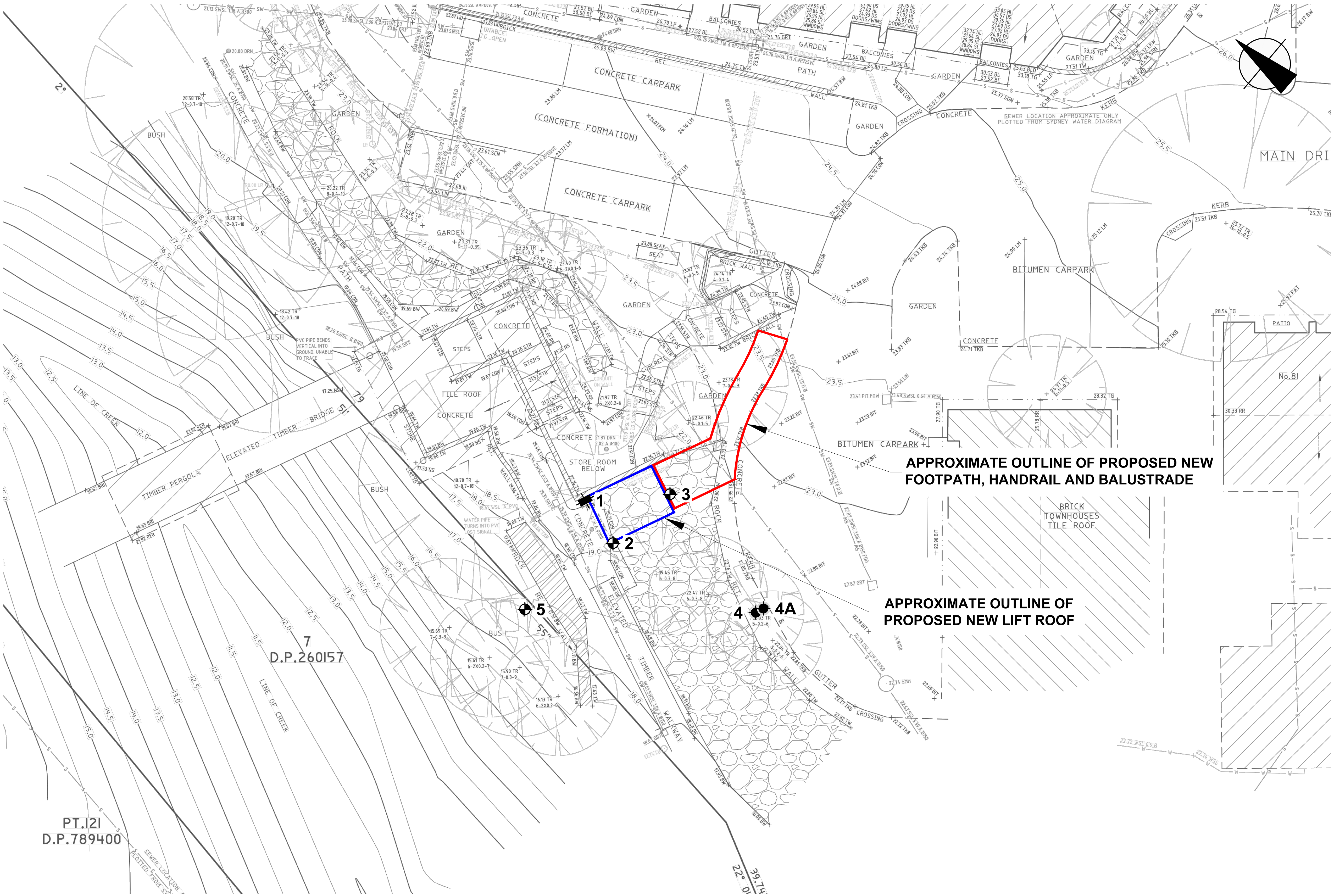
PLOT DATE: 9/10/2024 9:51:04 AM DWG FILE: J:\6F GEOTECHNICAL JOBS\37000'S\37080YF BAYVIEW\CAD\37080YF.DWG



LEGEND

- BOREHOLE AND DCP TEST
- DCP TEST
- TEST PIT, BOREHOLE AND DCP TEST

SCALE 1:200 @A3 METRES (0 2 4 6 8 10)

This plan should be read in conjunction with the JK Geotechnics report.

Title:	INVESTIGATION LOCATION PLAN
Location:	36-42 CABBAGE TREE ROAD, BAYVIEW, NSW
Report No:	37080YF
Figure No:	2

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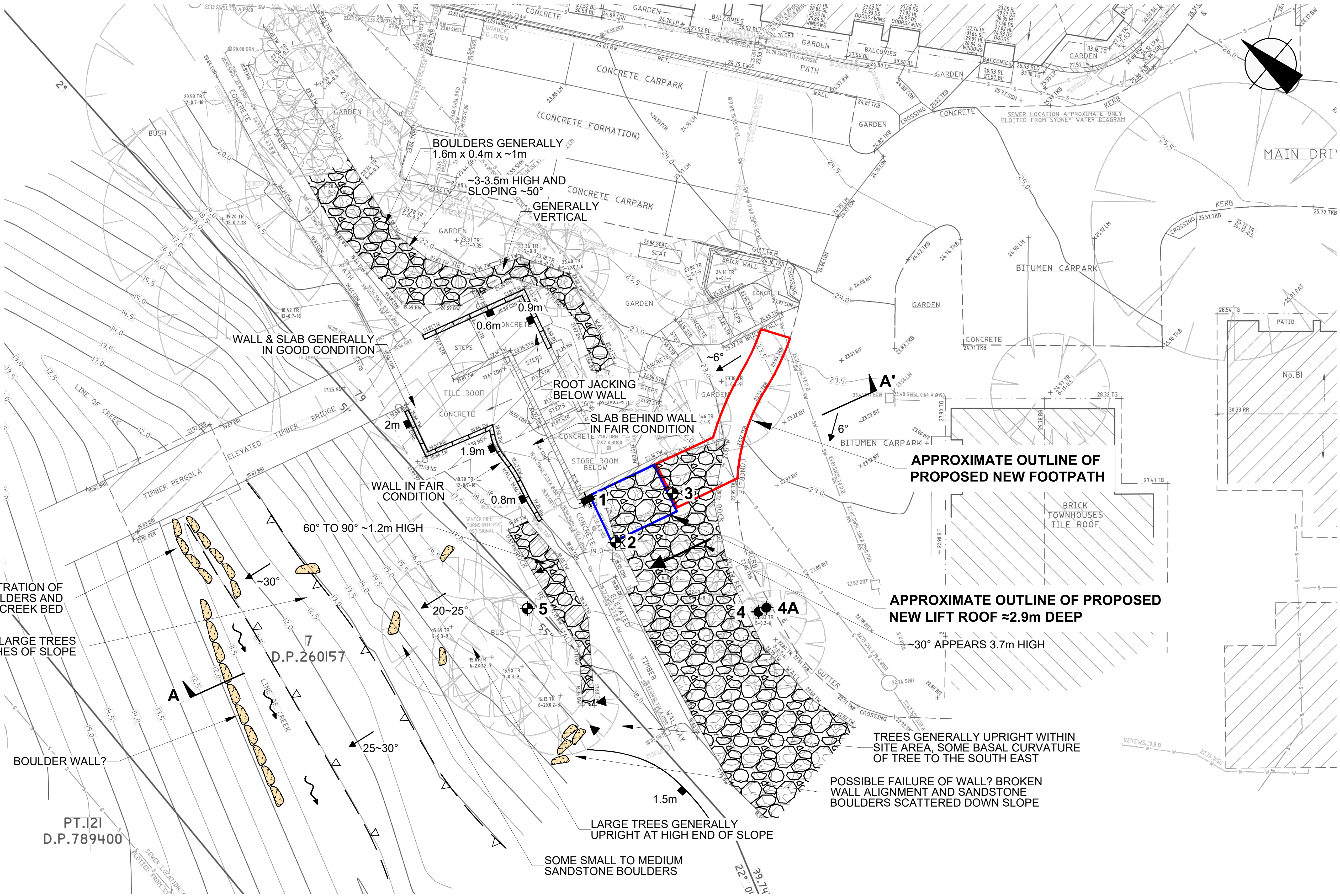
PLOT DATE: 9/10/2024 9:51:24 AM DWG FILE: J:\6F GEOTECHNICAL JOBS\37000'S\37080YF BAYVIEW\CAD\37080YF.DWG



LEGEND

- BOREHOLE AND DCP TEST
- DCP TEST
- TEST PIT, BOREHOLE AND DCP TEST
- A — A' CROSS SECTION
- SANDSTONE BOULDER

NOTES:
1. BOULDERS ARE SHOWN GRAPHICALLY AND ARE NOT TO SCALE.
2. SEWER DEPTH AS PER MARKINGS SPRAY PAINTED ON ROAD SURFACE.
3. MAPPING ONLY SHOWN WITHIN SUBJECT SITE VICINITY.
4. REFER TO FIGURE 5 FOR CROSS SECTION A-A'.
5. REFER TO FIGURE 6 FOR GEOTECHNICAL MAPPING SYMBOLS.

0 2 4 6 8 10
SCALE 1:200 @A3 METRES

This plan should be read in conjunction with the JK Geotechnics report.

Title:	GEOTECHNICAL SKETCH PLAN
Location:	36-42 CABBAGE TREE ROAD, BAYVIEW, NSW
Report No:	37080YF
Figure No:	3

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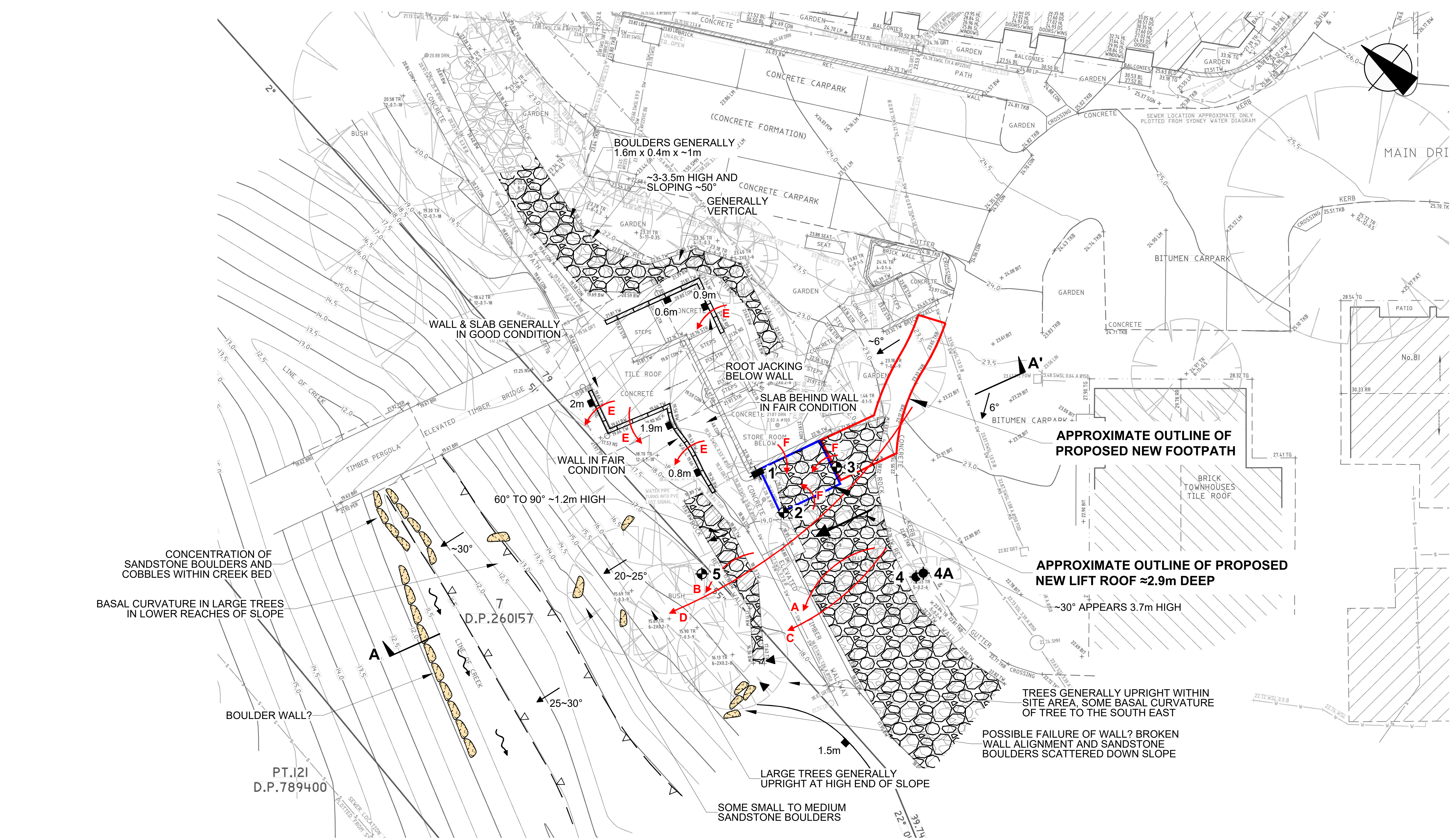
PLOT DATE: 16/10/2024 12:42:16 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\37000'S\37080YF BAYVIEW\CAD\37080YF.DWG



LEGEND

- BOREHOLE AND DCP TEST
- DCP TEST
- TEST PIT, BOREHOLE AND DCP TEST
- A—A' CROSS SECTION
- SANDSTONE BOULDER
- A GEOTECHNICAL HAZARD

NOTES:
1. BOULDERS ARE SHOWN GRAPHICALLY AND ARE NOT TO SCALE.
2. SEWER DEPTH AS PER MARKINGS SPRAY PAINTED ON ROAD SURFACE.
3. MAPPING ONLY SHOWN WITHIN SUBJECT SITE VICINITY.
4. REFER TO FIGURE 5 FOR CROSS SECTION A-A'.
5. REFER TO FIGURE 6 FOR GEOTECHNICAL MAPPING SYMBOLS.

0 2 4 6 8 10
SCALE 1:200 @A3 METRES

This plan should be read in conjunction with the JK Geotechnics report.

Title:	GEOTECHNICAL SKETCH PLAN SHOWING GEOTECHNICAL HAZARDS
Location:	36-42 CABBAGE TREE ROAD, BAYVIEW, NSW
Report No: 37080YF	Figure No: 4

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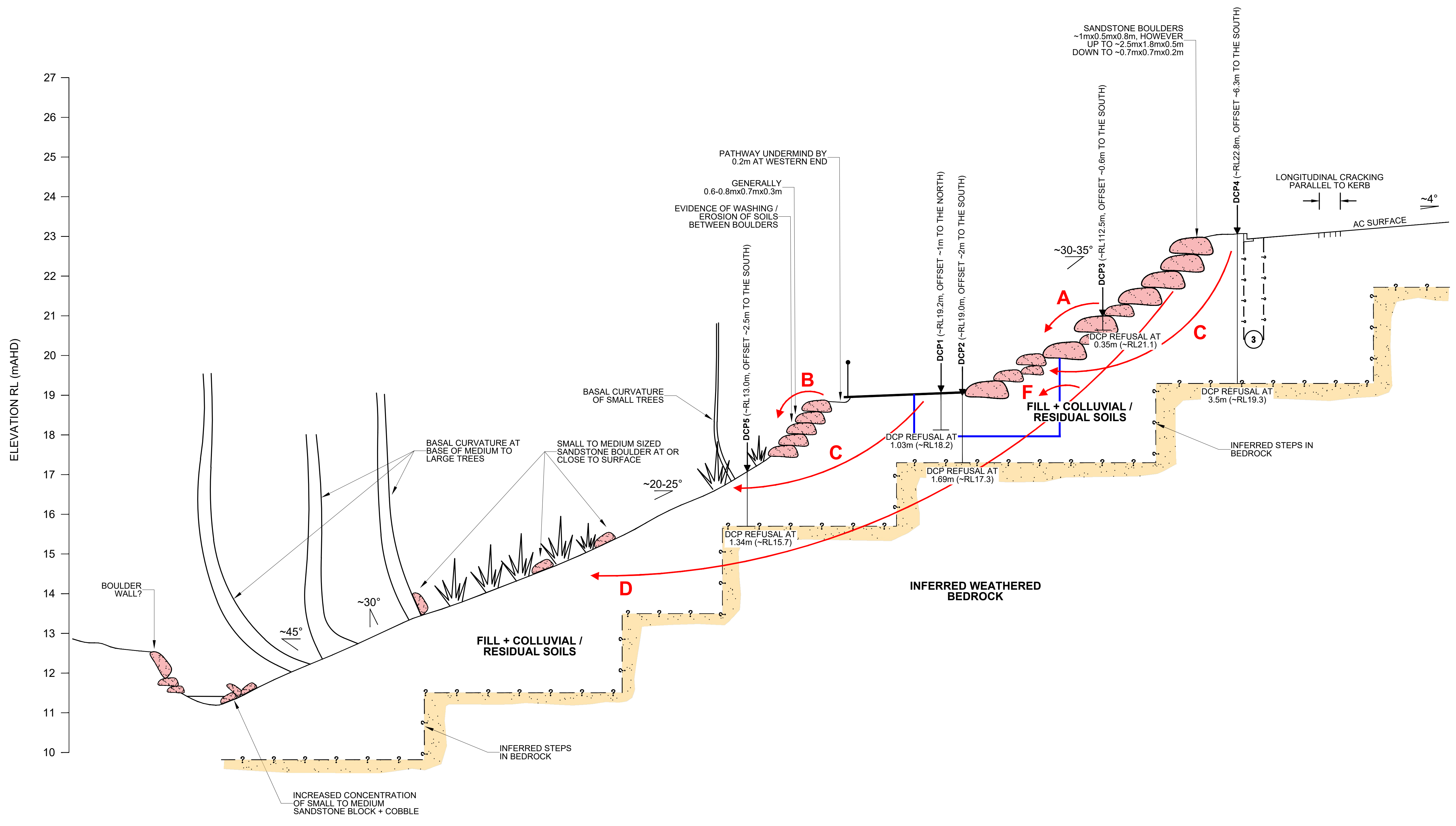

LEGEND

SANDSTONE BOULDER
INFERRED WEATHERED BEDROCK
INFERRED GEOTECHNICAL BOUNDARY
SEWER
GEOTECHNICAL HAZARD
PROPOSED BUSH LIFT AY ~RL17.9m

0 1 2 3 4 5
SCALE 1:100 @A3 METRES

This plan should be read in conjunction with the JK Geotechnics report.

Title: **CROSS SECTION A-A'**

Location: 36-42 CABBAGE TREE ROAD, BAYVIEW, NSW

Report No: 37080YF

Figure No: 5

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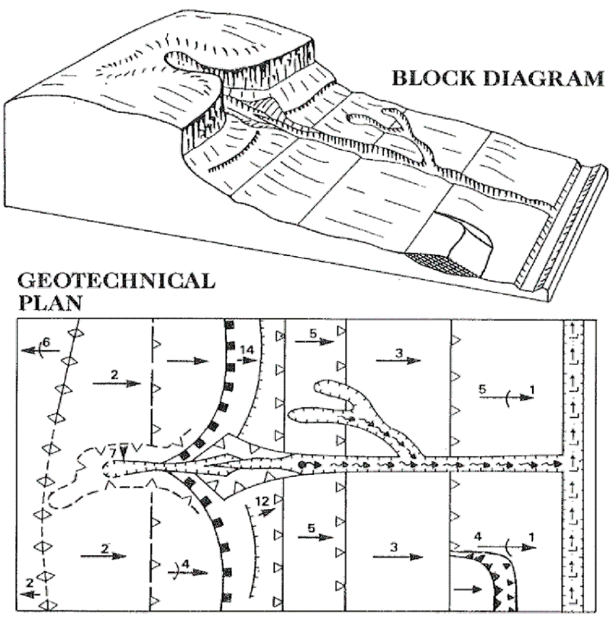
TOPOGRAPHY

Symbol	*Ground Profile*		
		convex	well defined or angular break of slope
		concave	
		convex	poorly defined or smooth change of slope
		concave	
breaks of slope			convex and concave too close together to allow the use of separate symbols
changes of slope			
sharp			ridge crest
rounded			
3 Cliff or escarpment or sharp break 40° or more (estimated height in metres)			
15 Uniform Slope			Slope direction and angle (Degrees)
10 Concave Slope			
8 Convex Slope			
Top			Cut or fill slope, arrows pointing down slope
Bottom			
Hummocky or irregular ground			

OTHER FEATURES

- Boulder
- Seepage/spring
- Swallow hole for runoff
- Natural water course
- Open drain, unlined
- Open drain, lined
- Fenceline
- Property boundary
- Dry Stone Wall
- J — J 200 Major joint in rock face (opening in millimetres)
- – T – T – 10 Tension crack (opening in millimetres)
- Masonry or concrete wall
- Ponding water
- Boggy or swampy area

EXAMPLE OF USE OF TOPOGRAPHIC SYMBOLS:



(After Gardiner, V & Dackombe, R. V. (1983), Geomorphological Field Manual; George Allen & Unwin).

This plan should be read in conjunction with the JK Geotechnics report.

Title:	GEOTECHNICAL MAPPING SYMBOLS
Location:	36-42 CABBAGE TREE ROAD, BAYVIEW, NSW
Report No: 37080YF	Figure No: 6
	JKGeotechnics

PLOT DATE: 9/10/2024 9:52:41 AM DWG FILE: J:\6F GEOTECHNICAL JOBS\37000'S\37080YF BAYVIEW\CAD\37080YF.DWG

APPENDIX A

LANDSLIDE RISK

MANAGEMENT

TERMINOLOGY

LANDSLIDE RISK MANAGEMENT

Definition of Terms and Landslide Risk

Risk Terminology	Description
Acceptable Risk	A risk for which, for the purposes of life or work, we are prepared to accept as it is with no regard to its management. Society does not generally consider expenditure in further reducing such risks justifiable.
Annual Exceedance Probability (AEP)	The estimated probability that an event of specified magnitude will be exceeded in any year.
Consequence	The outcomes or potential outcomes arising from the occurrence of a landslide expressed qualitatively or quantitatively, in terms of loss, disadvantage or gain, damage, injury or loss of life.
Elements at Risk	The population, buildings and engineering works, economic activities, public services utilities, infrastructure and environmental features in the area potentially affected by landslides.
Frequency	A measure of likelihood expressed as the number of occurrences of an event in a given time. See also 'Likelihood' and 'Probability'.
Hazard	A condition with the potential for causing an undesirable consequence (the landslide). The description of landslide hazard should include the location, volume (or area), classification and velocity of the potential landslides and any resultant detached material, and the likelihood of their occurrence within a given period of time.
Individual Risk to Life	The risk of fatality or injury to any identifiable (named) individual who lives within the zone impacted by the landslide; or who follows a particular pattern of life that might subject him or her to the consequences of the landslide.
Landslide Activity	The stage of development of a landslide; pre failure when the slope is strained throughout but is essentially intact; failure characterised by the formation of a continuous surface of rupture; post failure which includes movement from just after failure to when it essentially stops; and reactivation when the slope slides along one or several pre-existing surfaces of rupture. Reactivation may be occasional (eg. seasonal) or continuous (in which case the slide is 'active').
Landslide Intensity	A set of spatially distributed parameters related to the destructive power of a landslide. The parameters may be described quantitatively or qualitatively and may include maximum movement velocity, total displacement, differential displacement, depth of the moving mass, peak discharge per unit width, or kinetic energy per unit area.
Landslide Risk	The AGS Australian GeoGuide LR7 (AGS, 2007e) should be referred to for an explanation of Landslide Risk.
Landslide Susceptibility	The classification, and volume (or area) of landslides which exist or potentially may occur in an area or may travel or retrogress onto it. Susceptibility may also include a description of the velocity and intensity of the existing or potential landsliding.
Likelihood	Used as a qualitative description of probability or frequency.
Probability	A measure of the degree of certainty. This measure has a value between zero (impossibility) and 1.0 (certainty). It is an estimate of the likelihood of the magnitude of the uncertain quantity, or the likelihood of the occurrence of the uncertain future event. These are two main interpretations: (i) Statistical – frequency or fraction – The outcome of a repetitive experiment of some kind like flipping coins. It includes also the idea of population variability. Such a number is called an 'objective' or relative frequentist probability because it exists in the real world and is in principle measurable by doing the experiment.

Risk Terminology	Description
Probability *(continued)*	(ii) Subjective probability (degree of belief) – Quantified measure of belief, judgment, or confidence in the likelihood of an outcome, obtained by considering all available information honestly, fairly, and with a minimum of bias. Subjective probability is affected by the state of understanding of a process, judgment regarding an evaluation, or the quality and quantity of information. It may change over time as the state of knowledge changes.
Qualitative Risk Analysis	An analysis which uses word form, descriptive or numeric rating scales to describe the magnitude of potential consequences and the likelihood that those consequences will occur.
Quantitative Risk Analysis	An analysis based on numerical values of the probability, vulnerability and consequences and resulting in a numerical value of the risk.
Risk	A measure of the probability and severity of an adverse effect to health, property or the environment. Risk is often estimated by the product of probability x consequences. However, a more general interpretation of risk involves a comparison of the probability and consequences in a non-product form.
Risk Analysis	The use of available information to estimate the risk to individual, population, property, or the environment, from hazards. Risk analyses generally contain the following steps: scope definition, hazard identification and risk estimation.
Risk Assessment	The process of risk analysis and risk evaluation.
Risk Control or Risk Treatment	The process of decision-making for managing risk and the implementation or enforcement of risk mitigation measures and the re-evaluation of its effectiveness from time to time, using the results of risk assessment as one input.
Risk Estimation	The process used to produce a measure of the level of health, property or environmental risks being analysed. Risk estimation contains the following steps: frequency analysis, consequence analysis and their integration.
Risk Evaluation	The stage at which values and judgments enter the decision process, explicitly or implicitly, by including consideration of the importance of the estimated risks and the associated social, environmental and economic consequences, in order to identify a range of alternatives for managing the risks.
Risk Management	The complete process of risk assessment and risk control (or risk treatment).
Societal Risk	The risk of multiple fatalities or injuries in society as a whole: one where society would have to carry the burden of a landslide causing a number of deaths, injuries, financial, environmental and other losses.
Susceptibility	See 'Landslide Susceptibility'.
Temporal Spatial Probability	The probability that the element at risk is in the area affected by the landsliding, at the time of the landslide.
Tolerable Risk	A risk within a range that society can live with so as to secure certain net benefits. It is a range of risk regarded as non-negligible and needing to be kept under review and reduced further if possible.
Vulnerability	The degree of loss to a given element or set of elements within the area affected by the landslide hazard. It is expressed on a scale of 0 (no loss) to 1 (total loss). For property, the loss will be the value of the damage relative to the value of the property; for persons, it will be the probability that a particular life (the element at risk) will be lost, given the person(s) is affected by the landslide.

NOTE: Reference should be made to Figure A1 which shows the inter-relationship of many of these terms and the relevant portion of Landslide Risk Management.

Reference should also be made to the paper referenced below for Landslide Terminology and more detailed discussion of the above terminology.

This appendix is an extract from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in Australian Geomechanics, Vol 42, No 1, March 2007, which discusses the matter more fully.

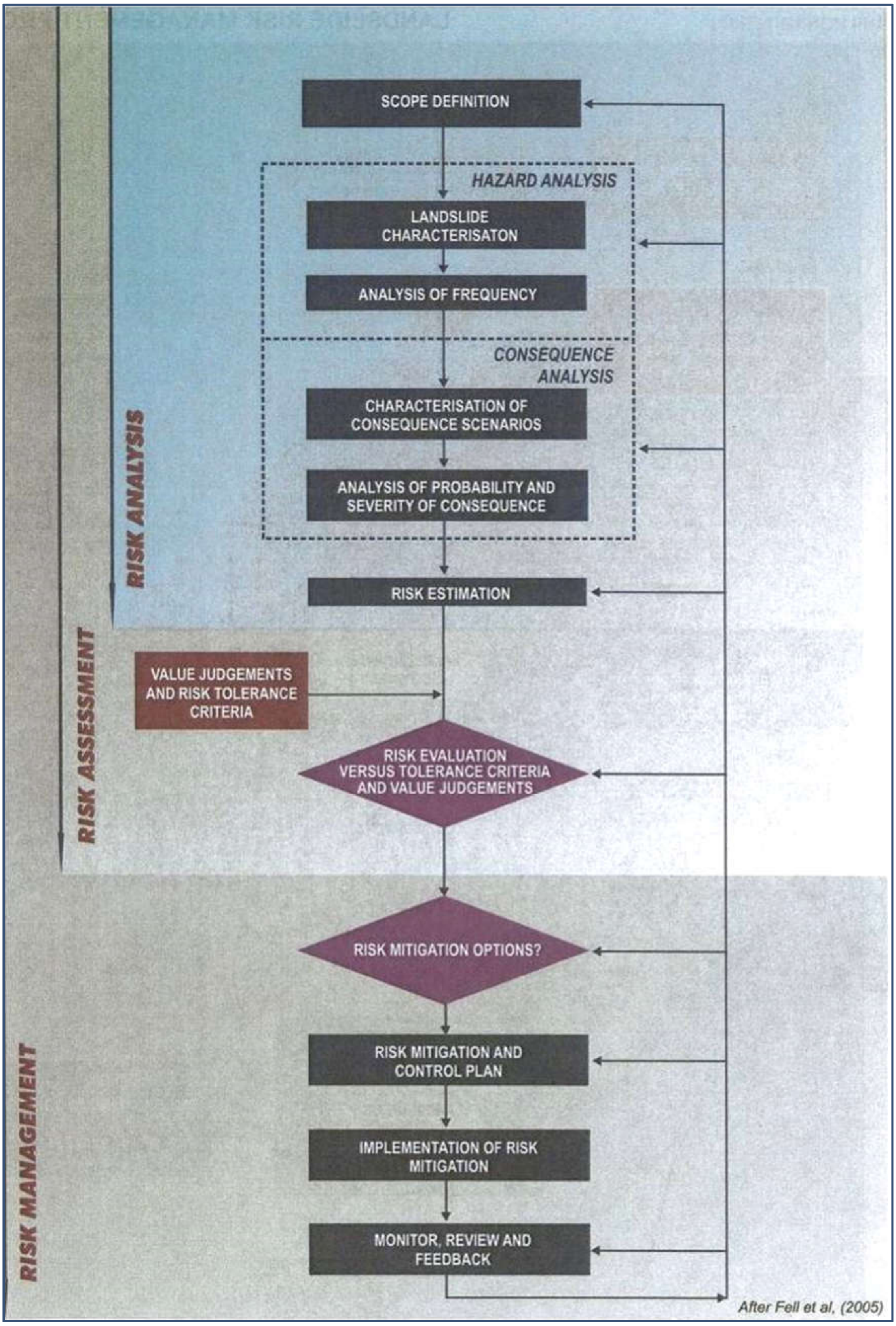


FIGURE A1: Flowchart for Landslide Risk Management.

This figure is an extract from GUIDELINE FOR LANDSLIDE SUSCEPTIBILITY, HAZARD AND RISK ZONING FOR LAND USE PLANNING, as presented in Australian Geomechanics Vol 42, No 1, March 2007, which discusses the matter more fully.

TABLE A1: LANDSLIDE RISK ASSESSMENT
QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

QUALITATIVE MEASURES OF LIKELIHOOD

Approximate Annual Probability		Implied Indicative Landslide Recurrence Interval		Description	Descriptor	Level
Indicative Value	Notional Boundary					
10^{-1}	5×10^{-2}	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10^{-2}	5×10^{-3}	100 years	200 years	The event will probably occur under adverse conditions over the design life.	LIKELY	B
10^{-3}	5×10^{-4}	1000 years	2000 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10^{-4}	5×10^{-5}	10,000 years	20,000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10^{-5}	5×10^{-2}	100,000 years	200,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10^{-6}		1,000,000 years		The event is inconceivable or fanciful over the design life.	BARELY CREDIBLE	F

Note: (1) The table should be used from left to right; use Approximate Annual Probability or Description to assign Descriptor, not *vice versa*.

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

Approximate cost of Damage		Description	Descriptor	Level
Indicative Value	Notional Boundary			
200%	100%	Structure(s) completely destroyed and/or large scale damage requiring major engineering works for stabilisation. Could cause at least one adjacent property major consequence damage.	CATASTROPHIC	1
60%	40%	Extensive damage to most of structure, and/or extending beyond site boundaries requiring significant stabilisation works. Could cause at least one adjacent property medium consequence damage.	MAJOR	2
20%	10%	Moderate damage to some of structure, and/or significant part of site requiring large stabilisation works. Could cause at least one adjacent property minor consequence damage.	MEDIUM	3
5%	1%	Limited damage to part of structure, and/or part of site requiring some reinstatement stabilisation works.	MINOR	4
0.5%		Little damage. (Note for high probability event (Almost Certain), this category may be subdivided at a notional boundary of 0.1%. See Risk Matrix.)	INSIGNIFICANT	5

Notes: (2) The Approximate Cost of Damage is expressed as a percentage of market value, being the cost of the improved value of the unaffected property which includes the land plus the unaffected structures.

(3) The Approximate Cost is to be an estimate of the direct cost of the damage, such as the cost of reinstatement of the damaged portion of the property (land plus structures), stabilisation works required to render the site to tolerable risk level for the landslide which has occurred and professional design fees, and consequential costs such as legal fees, temporary accommodation. It does not include additional stabilisation works to address other landslides which may affect the property.

(4) The table should be used from left to right; use Approximate Cost of Damage or Description to assign Descriptor, not vice versa.

Extract from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in Australian Geomechanics, Vol 42, No 1, March 2007, which discusses the matter more fully.

TABLE A1: LANDSLIDE RISK ASSESSMENT
QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY (continued)

QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY

LIKELIHOOD		CONSEQUENCES TO PROPERTY (With Indicative Approximate Cost of Damage)				
	Indicative Value of Approximate Annual Probability	1: CATASTROPHIC 200%	2: MAJOR 60%	3: MEDIUM 20%	4: MINOR 5%	5: INSIGNIFICANT 0.5%
A – ALMOST CERTAIN	10^{-1}	VH	VH	VH	H	M or **L** (5)
B - LIKELY	10^{-2}	VH	VH	H	M	**L**
C - POSSIBLE	10^{-3}	VH	H	M	M	**VL**
D - UNLIKELY	10^{-4}	H	M	**L**	**L**	**VL**
E - RARE	10^{-5}	M	**L**	**L**	**VL**	**VL**
F - BARELY CREDIBLE	10^{-6}	**L**	**VL**	**VL**	**VL**	**VL**

Notes: (5) Cell A5 may be subdivided such that a consequence of less than 0.1% is Low Risk.

(6) When considering a risk assessment it must be clearly stated whether it is for existing conditions or with risk control measures which may not be implemented at the current time.

RISK LEVEL IMPLICATIONS

Risk Level		Example Implications (7)
VH	VERY HIGH RISK	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work likely to cost more than value of the property.
H	HIGH RISK	Unacceptable without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to Low. Work would cost a substantial sum in relation to the value of the property.
M	MODERATE RISK	May be tolerated in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment options to reduce the risk to Low. Treatment options to reduce to Low risk should be implemented as soon as practicable.
L	LOW RISK	Usually acceptable to regulators. Where treatment has been required to reduce the risk to this level, ongoing maintenance is required.
VL	VERY LOW RISK	Acceptable. Manage by normal slope maintenance procedures.

Note: (7) The implications for a particular situation are to be determined by all parties to the risk assessment and may depend on the nature of the property at risk; these are only given as a general guide.

Extract from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in Australian Geomechanics, Vol 42, No 1, March 2007, which discusses the matter more fully.

AUSTRALIAN GEOGUIDE LR2 (LANDSLIDES)

What is a Landslide?

Any movement of a mass of rock, debris, or earth, down a slope, constitutes a "landslide". Landslides take many forms, some of which are illustrated. More information can be obtained from Geoscience Australia, or by visiting its Australian landslide Database at www.ga.gov.au/urban/factsheets/landslide.jsp. Aspects of the impact of landslides on buildings are dealt with in the book "Guideline Document Landslide Hazards" published by the Australian Building Codes Board and referenced in the Building Code of Australia. This document can be purchased over the internet at the Australian Building Codes Board's website www.abcb.gov.au.

Landslides vary in size. They can be small and localised or very large, sometimes extending for kilometres and involving millions of tonnes of soil or rock. It is important to realise that even a 1 cubic metre boulder of soil, or rock, weighs at least 2 tonnes. If it falls, or slides, it is large enough to kill a person, crush a car, or cause serious structural damage to a house. The material in a landslide may travel downhill well beyond the point where the failure first occurred, leaving destruction in its wake. It may also leave an unstable slope in the ground behind it, which has the potential to fall again, causing the landslide to extend (regress) uphill, or expand sideways. For all these reasons, both "potential" and "actual" landslides must be taken very seriously. The present a real threat to life and property and require proper management.

Identification of landslide risk is a complex task and must be undertaken by a geotechnical practitioner (GeoGuide LR1) with specialist experience in slope stability assessment and slope stabilisation.

What Causes a Landslide?

Landslides occur as a result of local geological and groundwater conditions, but can be exacerbated by inappropriate development (GeoGuide LR8), exceptional weather, earthquakes and other factors. Some slopes and cliffs never seem to change, but are actually on the verge of failing. Others, often moderate slopes (Table 1), move continuously, but so slowly that it is not apparent to a casual observer. In both cases, small changes in conditions can trigger a landslide with series consequences. Wetting up of the ground (which may involve a rise in groundwater table) is the single most important cause of landslides (GeoGuide LR5). This is why they often occur during, or soon after, heavy rain. Inappropriate development often results in small scale landslides which are very expensive in human terms because of the proximity of housing and people.

Does a Landslide Affect You?

Any slope, cliff, cutting, or fill embankment may be a hazard which has the potential to impact on people, property, roads and services. Some tell-tale signs that might indicate that a landslide is occurring are listed below:

- Open cracks, or steps, along contours
- Groundwater seepage, or springs
- Bulging in the lower part of the slope
- Hummocky ground
- trees leaning down slope, or with exposed roots
- debris/fallen rocks at the foot of a cliff
- tilted power poles, or fences
- cracked or distorted structures

These indications of instability may be seen on almost any slope and are not necessarily confined to the steeper ones (Table 1). Advice should be sought from a geotechnical practitioner if any of them are observed. Landslides do not respect property boundaries. As mentioned above they can "run-out" from above, "regress" from below, or expand sideways, so a landslide hazard affecting your property may actually exist on someone else's land.

Local councils are usually aware of slope instability problems within their jurisdiction and often have specific development and maintenance requirements. **Your local council is the first place to make enquiries if you are responsible for any sort of development or own or occupy property on or near sloping land or a cliff.**

TABLE 1 – Slope Descriptions

Appearance	Slope Angle	Maximum Gradient	Slope Characteristics
Gentle	0° - 10°	1 on 6	Easy walking.
Moderate	10° - 18°	1 on 3	Walkable. Can drive and manoeuvre a car on driveway.
Steep	18° - 27°	1 on 2	Walkable with effort. Possible to drive straight up or down roughened concrete driveway, but cannot practically manoeuvre a car.
Very Steep	27° - 45°	1 on 1	Can only climb slope by clutching at vegetation, rocks, etc.
Extreme	45° - 64°	1 on 0.5	Need rope access to climb slope.
Cliff	64° - 84°	1 on 0.1	Appears vertical. Can abseil down.
Vertical or Overhang	84° - 90±°	Infinite	Appears to overhang. Abseiler likely to lose contact with the face.

Some typical landslides which could affect residential housing are illustrated below:

Rotational or circular slip failures (Figure 1) - can occur on moderate to very steep soil and weathered rock slopes (Table 1). The sliding surface of the moving mass tends to be deep seated. Tension cracks may open at the top of the slope and bulging may occur at the toe. The ground may move in discrete "steps" separated by long periods without movement. More rapid movement may occur after heavy rain.

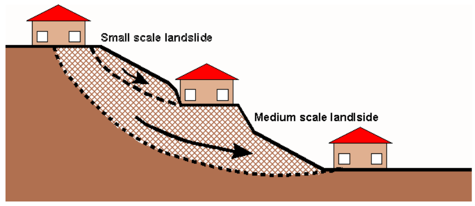


Figure 1

Translational slip failures (Figure 2) - tend to occur on moderate to very steep slopes (Table 1) where soil, or weak rock, overlies stronger strata. The sliding mass is often relatively shallow. It can move, or deform slowly (creep) over long periods of time. Extensive linear cracks and hummocks sometimes form along the contours. The sliding mass may accelerate after heavy rain.

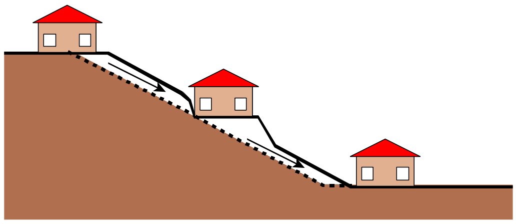


Figure 2

Wedge failures (Figure 3) - normally only occur on extreme slopes, or cliffs (Table 1), where discontinuities in the rock are inclined steeply downwards out of the face.

Rock falls (Figure 3) - tend to occur from cliffs and overhangs (Table 1).

Cliffs may remain, apparently unchanged, for hundreds of years. Collections of boulders at the foot of a cliff may indicate that rock falls are ongoing. Wedge failures and rock falls do not "creep". Familiarity with a particular local situation can instil a false sense of security since failure, when it occurs, is usually sudden and catastrophic.

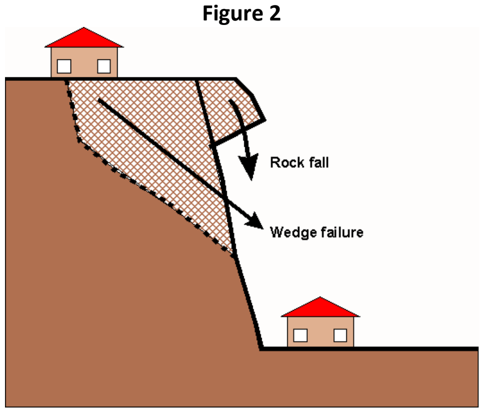


Figure 3

Debris flows and mud slides (Figure 4) - may occur in the foothills of ranges, where erosion has formed valleys which slope down to the plains below. The valley bottoms are often lined with loose eroded material (debris) which can "flow" if it becomes saturated during and after heavy rain. Debris flows are likely to occur with little warning; they travel a long way and often involve large volumes of soil. The consequences can be devastating.

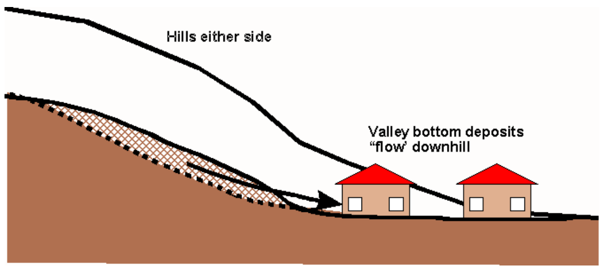


Figure 4

More information relevant to your particular situation may be found in other Australian GeoGuides:

- GeoGuide LR1 - Introduction
- GeoGuide LR3 - Soil Slopes
- GeoGuide LR4 - Rock Slopes
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR7 - Landslide Risk
- GeoGuide LR8 - Hillside Construction
- GeoGuide LR9 - Effluent & Surface Water Disposal
- GeoGuide LR10 - Coastal Landslides
- GeoGuide LR11 - Record Keeping

The Australian GeoGuides (LR series) are a set of publications intended for property owners; local councils; planning authorities; developers; insurers; lawyers and, in fact, anyone who lives with, or has an interest in, a natural or engineered slope, a cutting, or an excavation. They are intended to help you understand why slopes and retaining structures can be a hazard and what can be done with appropriate professional advice and local council approval (if required) to remove, reduce, or minimise the risk they represent. The GeoGuides have been prepared by the Australian Geomechanics Society, a specialist technical society within Engineers Australia, the national peak body for all engineering disciplines in Australia, whose members are professional geotechnical engineers and engineering geologists with a particular interest in ground engineering. The GeoGuides have been funded under the Australian governments' National Disaster Mitigation Program.

AUSTRALIAN GEOGUIDE LR7 (LANDSLIDE RISK)

Concept of Risk

Risk is a familiar term, but what does it really mean? It can be defined as *"a measure of the probability and severity of an adverse effect to health, property, or the environment."* This definition may seem a bit complicated. In relation to landslides, geotechnical practitioners (see GeoGuide LR1) are required to assess risk in terms of the likelihood that a particular landslide will occur and the possible consequences. This is called landslide risk assessment. The consequences of a landslide are many and varied, but our concerns normally focus on loss of, or damage to, property and loss of life.

Landslide Risk Assessment

Some local councils in Australia are aware of the potential for landslides within their jurisdiction and have responded by designating specific "landslide hazard zones". Development in these areas is normally covered by special regulations. If you are contemplating building, or buying an existing house, particularly in a hilly area, or near cliffs, then go first for information to your local council.

<u>Landslide risk assessment must be undertaken by a geotechnical practitioner.</u> It may involve visual inspection, geological mapping, geotechnical investigation and monitoring to identify:

- potential landslides (there may be more than one that could impact on your site);
- the likelihood that they will occur;
- the damage that could result;
- the cost of disruption and repairs; and
- the extent to which lives could be lost.

Risk assessment is a predictive exercise, but since the ground and the processes involved are complex, prediction tends to lack precision. If you commission a landslide risk assessment for a particular site you should expect to receive a report prepared in accordance with current professional guidelines and in a form that is acceptable to your local council, or planning authority.

Risk to Property

Table 1 indicates the terms used to describe risk to property. Each risk level depends on an assessment of how likely a landslide is to occur and its consequences in dollar terms. "Likelihood" is the chance of it happening in any one year, as indicated in Table 2. "Consequences" are related to the cost of the repairs and temporary loss of use if the landslide occurs. These two factors are combined by the geotechnical practitioner to determine the Qualitative Risk.

TABLE 2 – LIKELIHOOD

Likelihood	Annual Probability
Almost Certain	1:10
Likely	1:100
Possible	1:1,000
Unlikely	1:10,000
Rare	1:100,000
Barely credible	1:1,000,000

The terms "unacceptable", "may be tolerable" etc. in Table 1 indicate how most people react to an assessed risk level. However, some people will always be more prepared, or better able, to tolerate a higher risk level than others.

Some local councils and planning authorities stipulate a maximum tolerable risk level of risk to property for developments within their jurisdictions. In these situations the risk must be assessed by a geotechnical practitioner. If stabilisation works are needed to meet the stipulated requirements these will normally have to be carried out as part of the development, or consent will be withheld.

TABLE 1 – RISK TO PROPERTY

Qualitative Risk		Significance - Geotechnical engineering requirements
Very high	VH	**Unacceptable** without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low. May be too expensive and not practical. Work likely to cost more than the value of the property.
High	H	**Unacceptable** without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to acceptable level. Work would cost a substantial sum in relation to the value of the property.
Moderate	M	**May be tolerated** in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment options to reduce the risk to Low. Treatment options to reduce to Low risk should be implemented as soon as possible.
Low	L	**Usually acceptable** to regulators. Where treatment has been needed to reduce the risk to this level, ongoing maintenance is required.
Very Low	VL	**Acceptable**. Manage by normal slope maintenance procedures.

Risk to Life

Most of us have some difficulty grappling with the concept of risk and deciding whether, or not, we are prepared to accept it. However, without doing any sort of analysis, or commissioning a report from an "expert", we all take risks every day. One of them is the risk of being killed in an accident. This is worth thinking about, because it tells us a lot about ourselves and can help to put an assessed risk into a meaningful context. By identifying activities that we either are, or are not, prepared to engage in, we can get some indication of the maximum level of risk that we are prepared to take. This knowledge can help us to decide whether we really are able to accept a particular risk, or to tolerate a particular likelihood of loss, or damage, to our property (Table 2).

In Table 3, data from NSW for the years 1998 to 2002, and other sources, is presented. A risk of 1 in 100,000 means that, in any one year, 1 person is killed for every 100,000 people undertaking that particular activity. The NSW data assumes that the whole population undertakes the activity. That is, we are all at risk of being killed in a fire, or of choking on our food, but it is reasonable to assume that only people who go deep sea fishing run a risk of being killed while doing it.

It can be seen that the risks of dying as a result of falling, using a motor vehicle, or engaging in water-related activities (including bathing) are all greater than 1:100,000 and yet few people actively avoid situations where these risks are present. Some people are averse to flying and yet it represents a lower risk than choking to death on food. The data also indicate that, even when the risk of dying as a consequence of a particular event is very small, it could still happen to any one of us today. If this were not so, there would be no risk at all and clearly that is not the case.

In NSW, the planning authorities consider that 1:1,000,000 is the maximum tolerable risk for domestic housing built near an obvious hazard, such as a chemical factory. Although not specifically considered in the NSW guidelines there is little difference between the hazard presented by a neighbouring factory and a landslide: both have the capacity to destroy life and property and both are always present.

TABLE 3 – RISK TO LIFE

Risk (deaths per participant per year)	**Activity/Event Leading to Death** (NSW data unless noted)
1:1,000	Deep sea fishing (UK)
1:1,000 to 1:10,000	Motor cycling, horse riding, ultra-light flying (Canada)
1:23,000	Motor vehicle use
1:30,000	Fall
1:70,000	Drowning
1:180,000	Fire/burn
1:660,000	Choking on food
1:1,000,000	Scheduled airlines (Canada)
1:2,300,000	Train travel
1:32,000,000	Lightning strike

More information relevant to your particular situation may be found in other Australian GeoGuides:

- GeoGuide LR1 - Introduction
- GeoGuide LR3 - Soil Slopes
- GeoGuide LR4 - Rock Slopes
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
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The Australian GeoGuides (LR series) are a set of publications intended for property owners; local councils; planning authorities; developers; insurers; lawyers and, in fact, anyone who lives with, or has an interest in, a natural or engineered slope, a cutting, or an excavation. They are intended to help you understand why slopes and retaining structures can be a hazard and what can be done with appropriate professional advice and local council approval (if required) to remove, reduce, or minimise the risk they represent. The GeoGuides have been prepared by the Australian Geomechanics Society, a specialist technical society within Engineers Australia, the national peak body for all engineering disciplines in Australia, whose members are professional geotechnical engineers and engineering geologists with a particular interest in ground engineering. The GeoGuides have been funded under the Australian governments' National Disaster Mitigation Program.

APPENDIX B

SOME GUIDELINES

FOR

HILLSIDE CONSTRUCTION

SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

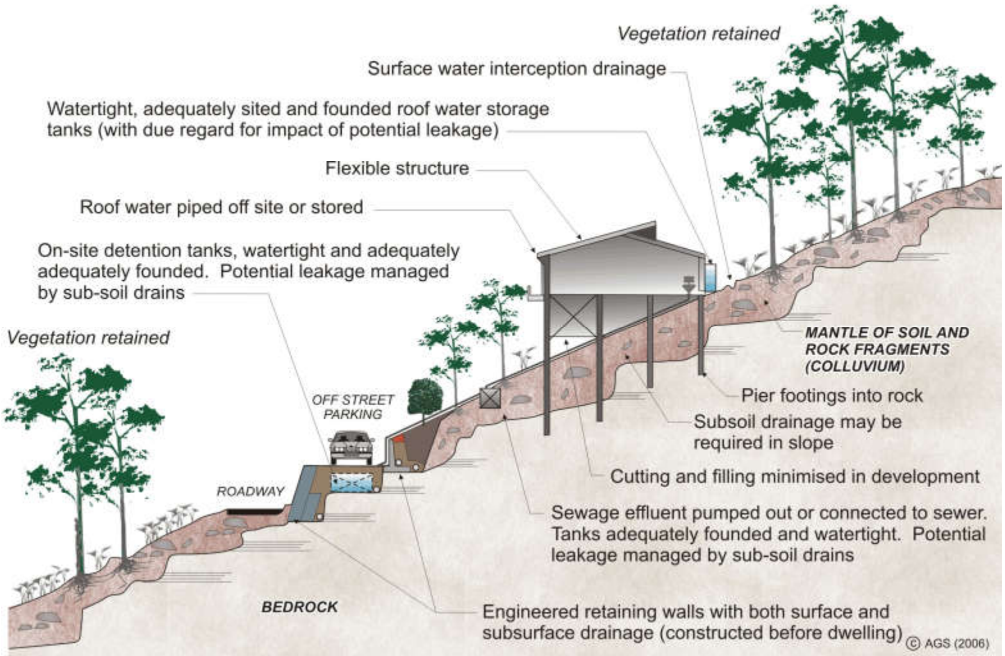
	GOOD ENGINEERING PRACTICE	POOR ENGINEERING PRACTICE
ADVICE		
GEOTECHNICAL ASSESSMENT	Obtain advice from a qualified, experienced geotechnical consultant at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.
PLANNING		
SITE PLANNING	Having obtained geotechnical advice, plan the development with the risk arising from the identified hazards and consequences in mind.	Plan development without regard for the Risk.
DESIGN AND CONSTRUCTION		
HOUSE DESIGN	Use flexible structures which incorporate properly designed brickwork, timber or steel frames, timber or panel cladding. Consider use of split levels. Use decks for recreational areas where appropriate.	Floor plans which require extensive cutting and filling. Movement intolerant structures.
SITE CLEARING	Retain natural vegetation wherever practicable.	Indiscriminately clear the site.
ACCESS & DRIVEWAYS	Satisfy requirements below for cuts, fills, retaining walls and drainage. Council specifications for grades may need to be modified. Driveways and parking areas may need to be fully supported on piers.	Excavate and fill for site access before geotechnical advice.
EARTHWORKS	Retain natural contours wherever possible.	Indiscriminant bulk earthworks.
CUTS	Minimise depth. Support with engineered retaining walls or batter to appropriate slope. Provide drainage measures and erosion control.	Large scale cuts and benching. Unsupported cuts. Ignore drainage requirements.
FILLS	Minimise height. Strip vegetation and topsoil and key into natural slopes prior to filling. Use clean fill materials and compact to engineering standards. Batter to appropriate slope or support with engineered retaining wall. Provide surface drainage and appropriate subsurface drainage.	Loose or poorly compacted fill, which if it fails, may flow a considerable distance (including onto properties below). Block natural drainage lines. Fill over existing vegetation and topsoil. Include stumps, trees, vegetation, topsoil, boulders, building rubble etc. in fill.
ROCK OUTCROPS & BOULDERS	Remove or stabilise boulders which may have unacceptable risk. Support rock faces where necessary.	Disturb or undercut detached blocks or boulders.
RETAINING WALLS	Engineer design to resist applied soil and water forces. Found on bedrock where practicable. Provide subsurface drainage within wall backfill and surface drainage on slope above. Construct wall as soon as possible after cut/fill operation.	Construct a structurally inadequate wall such as sandstone flagging, brick or unreinforced blockwork. Lack of subsurface drains and weepholes.
FOOTINGS	Found within bedrock where practicable. Use rows of piers or strip footings oriented up and down slope. Design for lateral creep pressures if necessary. Backfill footing excavations to exclude ingress of surface water.	Found on topsoil, loose fill, detached boulders or undercut cliffs.
SWIMMING POOLS	Engineer designed. Support on piers to rock where practicable. Provide with under-drainage and gravity drain outlet where practicable. Design for high soil pressures which may develop on uphill side whilst there may be little or no lateral support on downhill side.	
DRAINAGE SURFACE	Provide at tops of cut and fill slopes. Discharge to street drainage or natural water courses. Provide generous falls to prevent blockage by siltation and incorporate silt traps. Line to minimise infiltration and make flexible where possible. Special structures to dissipate energy at changes of slope and/or direction.	Discharge at top of fills and cuts. Allow water to pond bench areas.
SUBSURFACE	Provide filter around subsurface drain. Provide drain behind retaining walls. Use flexible pipelines with access for maintenance. Prevent inflow of surface water.	Discharge of roof run-off into absorption trenches.
SEPTIC & SULLAGE	Usually requires pump-out or mains sewer systems; absorption trenches may be possible in some areas if risk is acceptable. Storage tanks should be water-tight and adequately founded.	Discharge sullage directly onto and into slopes. Use of absorption trenches without consideration of landslide risk.
EROSION CONTROL & LANDSCAPING	Control erosion as this may lead to instability. Revegetate cleared area.	Failure to observe earthworks and drainage recommendations when landscaping.
DRAWINGS AND SITE VISITS DURING CONSTRUCTION		
DRAWINGS	Building Application drawings should be viewed by a geotechnical consultant.	
SITE VISITS	Site visits by consultant may be appropriate during construction.	
INSPECTION AND MAINTENANCE BY OWNER		
OWNER'S RESPONSIBILITY	Clean drainage systems; repair broken joints in drains and leaks in supply pipes. Where structural distress is evident seek advice. If seepage observed, determine cause or seek advice on consequences.	

This table is extracted from PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT as presented in *Australian Geomechanics*, Vol 42, No 1, March 2007 which discusses the matter more fully.

AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

Sensible development practices are required when building on hillsides, particularly if the hillside has more than a low risk of instability (GeoGuide LR7). Only building techniques intended to maintain, or reduce, the overall level of landslide risk should be considered. Examples of good hillside construction practice are illustrated below.

EXAMPLES FOR **GOOD** HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES GOOD?

Roadways and parking areas - are paved and incorporate kerbs which prevent water discharging straight into the hillside (GeoGuide LR5).

Cuttings - are supported by retaining walls (GeoGuide LR6).

Retaining walls - are engineer designed to withstand the lateral earth pressures and surcharges expected, and include drains to prevent water pressures developing in the backfill. Where the ground slopes steeply down towards the high side of a retaining wall, the disturbing force (see GeoGuide LR6) can be two or more times that due to level ground. Retaining walls must be designed taking these forces into account.

Sewage - whether treated or not is either taken away in pipes or contained in properly founded tanks so it cannot soak into the ground.

Surface water - from roofs and other hard surfaces is piped away to a suitable discharge point rather than being allowed to infiltrate into the ground. Preferably, the discharge point will be in a natural creek where ground water exits, rather than enters, the ground. Shallow, lined, drains on the surface can fulfill the same purpose (GeoGuide LR5).

Surface loads - are minimised. No fill embankments have been built. The house is a lightweight structure. Foundation loads have been taken down below the level at which a landslide is likely to occur and, preferably, to rock. This sort of construction is probably not applicable to soil slopes (GeoGuide LR3). If you are uncertain whether your site has rock near the surface, or is essentially a soil slope, you should engage a geotechnical practitioner to find out.

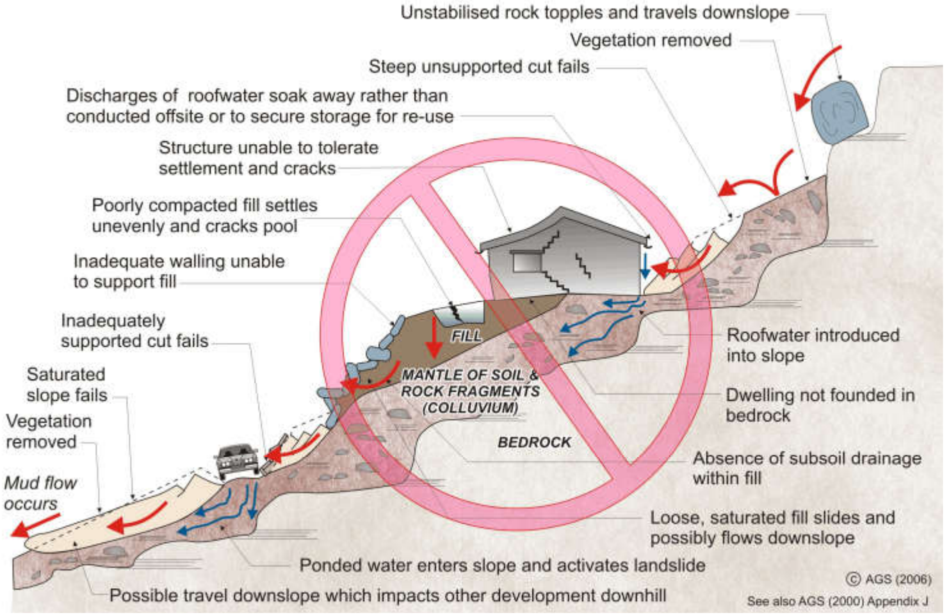
Flexible structures - have been used because they can tolerate a certain amount of movement with minimal signs of distress and maintain their functionality.

Vegetation clearance - on soil slopes has been kept to a reasonable minimum. Trees, and to a lesser extent smaller vegetation, take large quantities of water out of the ground every day. This lowers the ground water table, which in turn helps to maintain the stability of the slope. Large scale clearing can result in a rise in water table with a consequent increase in the likelihood of a landslide (GeoGuide LR5). An exception may have to be made to this rule on steep rock slopes where trees have little effect on the water table, but their roots pose a landslide hazard by dislodging boulders.

Possible effects of ignoring good construction practices are illustrated on page 2. Unfortunately, these poor construction practices are not as unusual as you might think and are often chosen because, on the face of it, they will save the developer, or owner, money. You should not lose sight of the fact that the cost and anguish associated with any one of the disasters illustrated, is likely to more than wipe out any apparent savings at the outset.

ADOPT GOOD PRACTICE ON HILLSIDE SITES

EXAMPLES FOR **POOR** HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES POOR?

Roadways and parking areas - are unsurfaced and lack proper table drains (gutters) causing surface water to pond and soaks into the ground.

Cut and fill - has been used to balance earthworks quantities and level the site leaving unstable cut faces and added large surface loads to the ground. Failure to compact the fill properly has led to settlement, which will probably continue for several years after completion. The house and pool have been built on the fill and have settled with it and cracked. Leakage from the cracked pool and the applied surface loads from the fill have combined to cause landslides.

Retaining walls - have been avoided, to minimise cost, and hand placed rock walls used instead. Without applying engineering design principles, the walls have failed to provide the required support to the ground and have failed, creating a very dangerous situation.

A heavy, rigid, house - has been built on shallow, conventional, footings. Not only has the brickwork cracked because of the resulting ground movements, but it has also become involved in a man-made landslide.

Soak-away drainage - has been used for sewage and surface water run-off from roofs and pavements. This water soaks into the ground and raises the water table (GeoGuide LR5). Subsoil drains that run along the contours should be avoided for the same reason. If felt necessary, subsoil drains should run steeply downhill in a chevron, or herringbone, pattern. This may conflict with the requirements for effluent and surface water disposal (GeoGuide LR9) and if so, you will need to seek professional advice.

Rock debris - from landslides higher up on the slope seems likely to pass through the site. Such locations are often referred to by geotechnical practitioners as "debris flow paths". Rock is normally even denser than ordinary fill, so even quite modest boulders are likely to weigh many tonnes and do a lot of damage once they start to roll. Boulders have been known to travel hundreds of metres downhill leaving behind a trail of destruction.

Vegetation - has been completely cleared, leading to a possible rise in the water table and increased landslide risk (GeoGuide LR5).

DON'T CUT CORNERS ON HILLSIDE SITES - OBTAIN ADVICE FROM A GEOTECHNICAL PRACTITIONER

More information relevant to your particular situation may be found in other Australian GeoGuides:

- GeoGuide LR1 - Introduction
- GeoGuide LR3 - Soil Slopes
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REPORT EXPLANATION NOTES

INTRODUCTION

These notes have been provided to amplify the geotechnical report in regard to classification methods, field procedures and certain matters relating to the Comments and Recommendations section. Not all notes are necessarily relevant to all reports.

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726:2017 *'Geotechnical Site Investigations'*. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	< 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2.36mm
Gravel	2.36 to 63mm
Cobbles	63 to 200mm
Boulders	> 200mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

Cohesive soils are classified on the basis of strength (consistency) either by use of a hand penetrometer, vane shear, laboratory testing and/or tactile engineering examination. The strength terms are defined as follows.

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	Strength not attainable – soil crumbles	

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'shale' is used to describe fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

SAMPLING

Sampling is carried out during drilling or from other excavations to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shrink-swell behaviour, strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils and 'weaker' bedrock if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for a large excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of limited reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock cuttings. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be assessed from the cuttings, together with some information from "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term 'mud' encompasses a range of products ranging from bentonite to polymers. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg. from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils, as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63.5kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

- In the case where full penetration is obtained with successive blow counts for each 150mm of, say, 4, 6 and 7 blows, as

  N = 13
  4, 6, 7

- In a case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm, as

  N > 30
  15, 30/40mm

The results of the test can be related empirically to the engineering properties of the soil.

A modification to the SPT is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as 'N_c' on the borehole logs, together with the number of blows per 150mm penetration.

Cone Penetrometer Testing (CPT) and Interpretation: The cone penetrometer is sometimes referred to as a Dutch Cone. The test is described in Australian Standard 1289.6.5.1–1999 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Static Cone Penetration Resistance of a Soil – Field Test using a Mechanical and Electrical Cone or Friction-Cone Penetrometer'*.

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck. The CPT does not provide soil sample recovery.

As penetration occurs (at a rate of approximately 20mm per second), the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

  The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between CPT and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of CPT values can be made to empirically derive modulus or compressibility values to allow calculation of foundation settlements.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

There are limitations when using the CPT in that it may not penetrate obstructions within any fill, thick layers of hard clay and very dense sand, gravel and weathered bedrock. Normally a 'dummy' cone is pushed through fill to protect the equipment. No information is recorded by the 'dummy' probe.

Flat Dilatometer Test: The flat dilatometer (DMT), also known as the Marchetti Dilometer comprises a stainless steel blade having a flat, circular steel membrane mounted flush on one side.

The blade is connected to a control unit at ground surface by a pneumatic-electrical tube running through the insertion rods. A gas tank, connected to the control unit by a pneumatic cable, supplies the gas pressure required to expand the membrane. The control unit is equipped with a pressure regulator, pressure gauges, an audio-visual signal and vent valves.

The blade is advanced into the ground using our CPT rig or one of our drilling rigs, and can be driven into the ground using an SPT hammer. As soon as the blade is in place, the membrane is inflated, and the pressure required to lift the membrane (approximately 0.1mm) is recorded. The pressure then required to lift the centre of the membrane by an additional 1mm is recorded. The membrane is then deflated before pushing to the next depth increment, usually 200mm down. The pressure readings are corrected for membrane stiffness.

The DMT is used to measure material index (I_D), horizontal stress index (K_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_o), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

The seismic dilatometer (SDMT) is the combination of the DMT with an add-on seismic module for the measurement of shear wave velocity (V_s). Using established correlations, the SDMT results can also be used to assess the small strain modulus (G_o).

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a 16mm diameter rod with a 20mm diameter cone end with a 9kg hammer dropping 510mm. The test is described in Australian Standard 1289.6.3.2–1997 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – 9kg Dynamic Cone Penetrometer Test'*.

The results are used to assess the relative compaction of fill, the relative density of granular soils, and the strength of cohesive soils. Using established correlations, the DCP test results can also be used to assess California Bearing Ratio (CBR).

Refusal of the DCP can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Vane Shear Test: The vane shear test is used to measure the undrained shear strength (C_u) of typically very soft to firm fine grained cohesive soils. The vane shear is normally performed in the bottom of a borehole, but can be completed from surface level, the bottom and sides of test pits, and on recovered undisturbed tube samples (when using a hand vane).

The vane comprises four rectangular blades arranged in the form of a cross on the end of a thin rod, which is coupled to the bottom of a drill rod string when used in a borehole. The size of the vane is dependent on the strength of the fine grained cohesive soils; that is, larger vanes are normally used for very low strength soils. For borehole testing, the size of the vane can be limited by the size of the casing that is used.

For testing inside a borehole, a device is used at the top of the casing, which suspends the vane and rods so that they do not sink under self-weight into the 'soft' soils beyond the depth at which the test is to be carried out. A calibrated torque head is used to rotate the rods and vane and to measure the resistance of the vane to rotation.

With the vane in position, torque is applied to cause rotation of the vane at a constant rate. A rate of 6° per minute is the common rotation rate. Rotation is continued until the soil is sheared and the maximum torque has been recorded. This value is then used to calculate the undrained shear strength. The vane is then rotated rapidly a number of times and the operation repeated until a constant torque reading is obtained. This torque value is used to calculate the remoulded shear strength. Where appropriate, friction on the vane rods is measured and taken into account in the shear strength calculation.

LOGS

The borehole or test pit logs presented herein are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on the frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will enable the most reliable assessment, but is not always practicable or possible to justify on economic grounds. In any case, the boreholes or test pits represent only a very small sample of the total subsurface conditions.

The terms and symbols used in preparation of the logs are defined in the following pages.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than 'straight line' variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

Where groundwater levels are measured in boreholes, there are several potential problems:

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes and may not be the same at the time of construction.
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must be washed out of the hole or 'reverted' chemically if reliable water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised at intervals ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg. bricks, steel, etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably assess the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

Reasonable care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.
- Details of the development that the Company could not reasonably be expected to anticipate.

If these occur, the Company will be pleased to assist with investigation or advice to resolve any problems occurring.

SITE ANOMALIES

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. Licence to use the documents may be revoked without notice if the Client is in breach of any obligation to make a payment to us.

REVIEW OF DESIGN

Where major civil or structural developments are proposed <u>or</u> where only a limited investigation has been completed <u>or</u> where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves an experienced geotechnical engineer/engineering geologist.

SITE INSPECTION

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

- FILL
- TOPSOIL
- CLAY (CL, CI, CH)
- SILT (ML, MH)
- SAND (SP, SW)
- GRAVEL (GP, GW)
- SANDY CLAY (CL, CI, CH)
- SILTY CLAY (CL, CI, CH)
- CLAYEY SAND (SC)
- SILTY SAND (SM)
- GRAVELLY CLAY (CL, CI, CH)
- CLAYEY GRAVEL (GC)
- SANDY SILT (ML, MH)
- PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK

- CONGLOMERATE
- SANDSTONE
- SHALE/MUDSTONE
- SILTSTONE
- CLAYSTONE
- COAL
- LAMINITE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 65% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–

Laboratory Classification Criteria

A well graded coarse grained soil is one for which the coefficient of uniformity $Cu > 4$ and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

$$C_U = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_C = \frac{(D_{30})^2}{D_{10}\,D_{60}}$$

Where D_{10}, D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

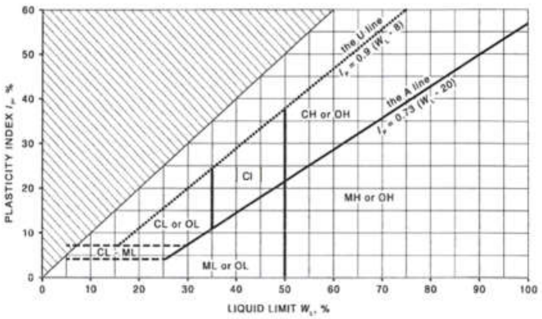
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Modified Casagrande Chart for Classifying Silts and Clays according to their Behaviour



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record		Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
		Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES	Sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
	ASB	Soil sample taken over depth indicated, for asbestos analysis.
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.
	SAL	Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25	Vane shear reading in kPa of undrained shear strength.
	PID = 100	Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL	Moisture content estimated to be greater than plastic limit.
	w ≈ PL	Moisture content estimated to be approximately equal to plastic limit.
	w < PL	Moisture content estimated to be less than plastic limit.
	w ≈ LL	Moisture content estimated to be near liquid limit.
	w > LL	Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D	DRY – runs freely through fingers.
	M	MOIST – does not run freely but no free water visible on soil surface.
	W	WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – unconfined compressive strength ≤ 25kPa.
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.
	Hd	HARD – unconfined compressive strength > 400kPa.
	Fr	FRIABLE – strength not attainable, soil crumbles.
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** / **SPT 'N' Value Range (Blows/300mm)**
	VL	VERY LOOSE – ≤ 15 – 0 – 4
	L	LOOSE – > 15 and ≤ 35 – 4 – 10
	MD	MEDIUM DENSE – > 35 and ≤ 65 – 10 – 30
	D	DENSE – > 65 and ≤ 85 – 30 – 50
	VD	VERY DENSE – > 85 – > 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

Log Column	Symbol	Definition
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Twin pronged tungsten carbide bit.
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.
	Soil Origin	The geological origin of the soil can generally be described as:
		RESIDUAL – soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock.
		EXTREMELY WEATHERED – soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock.
		ALLUVIAL – soil deposited by creeks and rivers.
		ESTUARINE – soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents.
		MARINE – soil deposited in a marine environment.
		AEOLIAN – soil carried and deposited by wind.
		COLLUVIAL – soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description 'slopewash' is used for thinner surficial deposits.
		LITTORAL – beach deposited soil.

Classification of Material Weathering

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

NOTE 1: The term 'Distinctly Weathered' is used where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock. 'Distinctly Weathered' is defined as follows: *'Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores'*. There is some change in rock strength.

Rock Material Strength Classification

			Guide to Strength	
Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Point Load Strength Index $I_{s(50)}$ (MPa)	Field Assessment
Very Low Strength	VL	0.6 to 2	0.03 to 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.
Low Strength	L	2 to 6	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium Strength	M	6 to 20	0.3 to 1	Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	H	20 to 60	1 to 3	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High Strength	VH	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High Strength	EH	> 200	> 10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Abbreviations Used in Defect Description

Cored Borehole Log Column		Symbol Abbreviation	Description
Point Load Strength Index		• 0.6	Axial point load strength index test result (MPa)
		x 0.6	Diametral point load strength index test result (MPa)
Defect Details	– Type	Be	Parting – bedding or cleavage
		CS	Clay seam
		Cr	Crushed/sheared seam or zone
		J	Joint
		Jh	Healed joint
		Ji	Incipient joint
		XWS	Extremely weathered seam
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres